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Water-Works Management and Maintenance

BY

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INTRODUCTION.

THE maintenance and operation of a system of water-works is often believed to be a purely business proposition requiring essentially a business management. Regarded in a broad and comprehensive sense this view may be correct, for a far-seeing business management would not overlook the purely technical or scientific considerations which are necessarily involved in the management of a modern water-works system. The questions involved do not relate solely to the sale of a commodity supplied in the form of a water service, but also deal with the quality of the water supplied and the design, construction, and operation of the physical property by and through which the service is rendered.

The selection of a water-supply drawn from an unpolluted source is highly desirable and inspires the confidence of the public in the management of water-works. This confidence, however, may be also secured when circumstances compel the use of a water drawn from polluted sources, provided the water be properly purified before use. Many natural waters contain more or less dissolved minerals, such as lime, magnesia, and iron, which are of themselves harmless, although in some instances objectionable when the water is required for some particular mechanical purpose. A soft water is always to be preferred for general use and is usually obtained from some surface source when a desirable selection can be made; but a moderately hard water is sometimes accepted for a public supply rather than incur the expense of the construction and operation of purification works. Turbidity and color, whether due to sediment mechanically suspended in water or to vegetable stain, are offensive even when present in a moderate degree, although they may not render the water unwholesome. The introduction of pathogenic bacteria into a water-supply by contamination by sewage or other wastes is often productive of disease, and the human system

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may be readily infected through the medium of drinking-water exposed to pollution of this nature. In consequence, sanitarians at the present time regard the quality of a water for hygienic purposes as closely associated with the bacterial life which it may contain.

Taking into consideration the many things which have to be regarded in the selection and purification of water-supplies, it is clear that science can be serviceable to a water-works management in many ways, and the advantage of this kind of service should become more and more apparent as communities increase and prosper. If the aid of science is necessary to select a source of supply free from dangerous pollution or to detect the presence of unobserved polluting influences, its aid is even more necessary in those cases where a source of supply, known to be polluted, requires thorough purification. It will not suffice to seek scientific assistance in such a case solely for the purpose of designing and constructing purification works, but it should also be retained for the purpose of insuring the satisfactory operation of these works and the preservation of the purity of the water after treatment. The safeguards of the public health in the way of constructed works need guardsmen to see that such works positively perform the functions expected of them at all times—a service which may yet have to be supplied through the State or Federal government.

From the foregoing it will appear that questions relating to the maintenance and operation of a system of water-works often depend for their proper solution upon the details of the original design, and the authors have deemed it advisable to outline to some extent in Part I the various methods by which a supply is secured.

Part II deals particularly with matters more closely connected with the ordinary routine management of a water-works plant.

In Part III consideration is given to questions which arise in the dealings of municipalities with private water companies. These questions have been brought to the attention of communities to a large extent during recent years by the trend toward municipal ownership of public utilities, and the principles involved in the fixing of water rates and the valuation of water-works property are presented in detail.

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WATER-WORKS MANAGEMENT AND MAINTENANCE.

PART I.

ON THE METHODS AND PRINCIPLES OF DEVELOPING, IMPROVING, AND STORING WATER-SUPPLIES.

CHAPTER I.

GROUND-WATER SUPPLY.

Quality.—There is a charm and satisfaction attached to the cool, sparkling, and refreshing draught from a natural spring which incites a warm and enduring friendship for it. The perfect transparency of the water and its comforting and refreshing effect offer no hint that between the sparkling spring and its origin, the rainfall, the water possesses a long and obscure history. Perhaps the very obscurity of the precise nature of this history gives substance to the water's enchanting influence. But hidden as it is and with no visible signs that that history could have been anything but good in its long and untraceable source through the ground, there is apparent to the observer no tangible ground upon which to base the slightest suspicion of its purity so long as it is agreeable both to taste and sight. We therefore detect in ground-water the nearest approach to what may be popularly considered an ideal drinking-water. In fact we find it above suspicion and reproach in rural districts, almost invariably of first consideration in communities desiring to acquire a public

water-supply, and often regarded technically as of the highest rank among the various sources of water-supply.

Sentiment and advertisement may stimulate a regard for a water of certain springs above that of other springs because the reputation thus gained usually centers around some medicinal qualities the water of such springs is supposed to possess. These special waters, the qualities of which are developed and sought out for special reasons and for special purposes, are not classed with the ordinary run of ground-waters available for the public water-supply.

The basis of popular acceptance of a ground-water is usually its appearance and general palatability, and as a rule its appearance neither presents nor suggests evidence of present or past pollution. However, a little reflection must tell us that the rainfall which washes the air and the surface of the ground must come in contact with contaminating influences to a greater or less degree, particularly in populous districts. Just how far we may encroach upon the domain of nature with our modern civilization and multiply instances of contamination upon her fair face without seriously polluting the rainfall, the source of all ground-water supply, is a question with which the water analyst and specialist is engaged. The further these investigations proceed along rational lines the cruder appear the past methods and means of determining and judging of the wholesomeness of a ground-water and the more secure appear the provisions of nature to remove the impurities collected by the rainfall even in populous districts. In fact it is exceedingly doubtful that we yet realize and fully appreciate the true character, scope, and security of these provisions of nature. While some forms of bacterial life may be a source of danger in water, it is well known that bacterial life of other forms is a source of security. For instance the portion of the rainfall which enters the ground, even though it may be grossly polluted by surface contact with decaying matter, is quickly relieved of its dissolved organic impurities by the bacterial life which infests the surface soils and thrives upon the food which percolating water conveys to it. The ideal geological condition for the natural purification of water is to be found in a plane of well-ventilated sandy soil upon which the rainfall remains quite

uniformly distributed until absorbed. A slow percolation through a comparatively few feet of such soil is sufficient for the purification of the rainfall. It is believed that pathogenic germs which water may have absorbed at the surface of the ground cannot survive a slow passage through such soil teeming with natural but to them antagonistic bacterial life. There is every reason to believe that a ground-water will be found quite as pure a comparatively few feet below the surface of a sandy soil as many feet below it, provided there are no more direct lines of communication than those through the voids of the sand.

Sanitary analysis of a ground-water tells of the existing condition of the water as exemplified by the samples analyzed, of the probable present and past relation of the water with putrescible organic matter, and of the possible contamination of the water from sources which cannot be detected by a personal inspection of the territory receiving the rainfall. It is indicative of the degree to which the process of natural purification has progressed. It deals with existing facts and conditions, and when regularly made affords direct evidence of changes in the characteristics of a water as they occur. It can give little reliable information of a danger threatening eventually to become a source of contamination.

A sanitary chemical analysis of a shallow ground-water concerns itself most with the existing state of the organic matter dissolved in the water analyzed, with the hardness of the water, and with the amount and condition of iron the water may contain, and with the total solids in solution.

The term albuminoid ammonia used in recording the results of a chemical analysis indicates the undecomposed organic matter remaining in a water, the terms free ammonia and nitrites are considered respectively measures of progressive decomposition of organic matter, and the term nitrates represents the final result of decomposition when the decomposed elements of the organic matter have united with some mineral in the water. The transformation in its completeness is from putrescible organic matter to a stable mineral compound. A water which possesses evidence of all the intermediate steps in this transformation is considered imperfectly purified and may be still subject to the agencies producing the change. The presence of albuminoid and free ammonias

in small quantities or in a stable condition, the absence of nitrites, and the presence of the nitrates in either large or small amounts afford evidence of complete purification.

Chlorine, usually present as common salt, has no direct chemical connection with the organic matter in water, is unchangeable and harmless, and is often regarded as an evidence of sewage pollution when the amount in the water exceeds that derived from natural sources.

A few years ago, before biology demonstrated bacteria in some form to be the active agents of water purification and the real element to be feared in a polluted water, the result of a chemical analysis showing abnormal amounts of chlorine and nitrates would have been considered sufficient evidence of impurity to condemn the water because of past association with organic impurity. This conclusion was based upon the consideration that suspicion should attach to a water showing evidence of past pollution because of a fear that the conditions of complete natural purification may not remain perfect, rather than upon a consideration of danger in the presence of the compounds themselves. At the present time the conditions surrounding the natural filtration of ground-water are better understood, and better understanding inspires greater confidence in the process of natural purification and offers a stronger assurance of the certainty with which this operation of nature is conducted even when water is exposed to much more pollution than that naturally encountered.

We are thus unable to draw rigidly the line of demarcation between a wholesome and an unwholesome water in so far as the quality of the water is revealed to us through the application of the science of chemistry alone. A knowledge of the physical characteristics of the territory from which the ground-water is supplied is often a more reliable guide than is the result of a chemical analysis alone. But both sources of information when fully developed should be mutually supporting.

The importance of the application of the science of bacteriology to the study of water-supply arises from the fact that drinking-water is a medium through which the bacilli of certain diseases may be imbibed and infection widely disseminated when the water is exposed to the pollution of infectious bacterial life without inter-

mediate provision for its destruction or removal. The most probable source of disease germs in water is direct sewage contamination, and therefore the bacteriologist searches for bacterial life characteristic of that known to exist in sewage in abundance. The presence in the water of sewage bacteria renders a water open to suspicion; the absence of this form of life allays suspicion even though the water may contain in small degree harmless forms of bacterial life native to water.

Attention is called to the following analysis to illustrate the effect which exposure of the rainfall to polluting matter upon the surface of the ground may have upon ground-water derived from springs and shallow wells.

(In parts per million.)

	Well 300 feet from Mississippi River.	Mississippi River water.	Spring Brook.
Total solid	264.00	329.00	126.0
Free ammonia	0.01	0.34	Trace
Albuminoid ammonia	0.014	0.394	0.028
Nitrites	None	....	None
Nitrates	9.6	0.02	0.028
Oxygen consumed	0.6	13.1	·
Chlorine	6.8	1.75	15.3

The analysis of the water of the well near the Mississippi River indicates past pollution of a considerable amount but thorough subsequent purification. The bacteria in the well were few, numbering 20 to 40 per cubic centimeter and of a harmless variety. The water is also noteworthy in the regard that the sample, though taken about 50 feet below the surface of the ground, contains less dissolved minerals than the water of the adjoining river. The catchment area supplying the well from which the sample of water analyzed was taken is a sandy plane extensively cultivated and heavily fertilized. Doubtless the origin of the nitrogen and abnormal chlorine in the well-water is the fertilizer which is used upon the surface of the ground.

The sample of water from the spring brook was taken near a spring which is supplied by rainfall upon a timbered, sandy, uncultivated catchment area. The analysis shows a small amount of organic pollution and accordingly is greatly in contrast with that of the sample supplied from the cultivated catchment area described in the preceding paragraph.

However, the character of the water from both the well and the spring brook could be substantially forecasted by survey of the physical characteristics of the respective catchment areas. Both waters were finally pronounced wholesome.

Careful survey of the physical characteristics of a catchment area feeding comparatively shallow water bearing sand and gravel beds taken in connection with the information furnished by chemical and bacterial analysis of the water itself constitutes a comprehensive sanitary survey, and when the results of the several examinations and analyses are interpreted with due mutual regard, we may reach substantially correct and safe conclusion as to the wholesomeness of a ground-water. A purely independent interpretation of the results of any one of the three elements referred to may lead to a wrong conclusion; for instance the results of the analysis of the well-water given in the preceding table, if interpreted from a chemical point of view alone in accordance with the so-called standard of purity of a potable water as laid down in some of the older books on water-supply, would certainly lead to a rejection of that water, although in fact the pollution indicated is purely a matter of the past state of the water. Similarly a purely physical examination may discover sources of pollution for the rainfall of such a character and so located as to arouse a suspicion that the polluting influence may extend to the subterranean waters and accordingly seek the assistance of both a chemical and bacteriological examination of the underground waters to either remove or confirm the suspicion.

In a general way it may be stated that polluting matter upon the surface of the ground cannot be carried by absorbed rainfall into the underground water by percolating through intervening soil without purification. The danger of pollution of this sort lies in a direct communication between the surface and the underground waters, as, for instance, when surface-water overflows or passes through the curb of an open well or well-developed fissure channels.

Polluting matter, particularly excrement deposited in covered excavations like the ordinary privy, is less exposed to natural purifying influences than similar matter upon the surface of the ground and accordingly is more to be feared, but the contami-

nating effect of such matter so deposited is confined to a much more restricted area than many people would suppose when the excrement rests on the subsoil. However, if the excrement rest upon fissured rock exposed by the excavation of the privy, the connection between the receptacle and underground water may be direct and pollution proportionally extended.

Water from deep wells possesses entirely different characteristics and cannot be regarded in the same category as shallow-well water. Usually such wells penetrate rock or alternate layers of porous and impervious materials and are supplied by the rainfall on remote catchment areas. As a rule the water from such wells is practically sterilized by a prolonged and slow process of filtration. A visual survey can scarcely be made of the catchment area except where the geological structure of the surrounding territory is known by extended surveys, and a bacterial analysis of the water is scarcely necessary because prolonged filtration deprives the water of bacterial life. A chemical examination is practically all that is needed, and this chiefly to determine the amount and character of dissolved mineral matter.

With regard to water of this class the subject can be concisely covered by quoting from the reports made in connection with the water-supply of Galveston, Texas:

"GALVESTON, TEXAS, November 19th, 1892.

"*To the Honorable Board of Commissioners of Water-works, Galveston, Texas.*

"Gentlemen: The following report is submitted, mainly from a sanitary standpoint, with respect to the new water-supply from artesian wells proposed for this city.

"The environment which gives character to deep-ground water is such that this class of water cannot be consistently compared with either surface or shallow-well waters.

"Nitrogen in several chemical combinations influences the determination of the quality of water for potable purposes more than any other element in chemical combinations, since it is a principal constituent of the decaying organic matter with which all natural waters come in contact. Whether the source

of this organic impurity in water is in decaying animal or vegetable matter, chemistry does not pretend to decide definitely. So much of the organic matter as is absorbed by rain falling in the open country is, as a rule, thoroughly oxidized and converted into the harmless inorganic nitrogen compound, nitrates, by intermittent filtration through porous surface soils in the presence of air. The nitrates in shallow-well waters, accompanied by small amounts of albuminoid and free ammonia, indicate a high degree of natural purification by intermittent filtration and the amount of previous contamination of water by dissolved organic matter. Should the nitrates be accompanied by a considerable amount of unstable albuminoid and free ammonia and nitrites there is evidence of recent contamination. The free ammonia, so called, is in itself harmless, and its presence in surface and shallow-well waters is simply indicative of a progressive natural process of purification of water that contains organic matter.

"The character of deep-well waters is entirely different. Because of the usual absence of free oxygen in these waters, nitrates once formed by intermittent filtration through well-aerated surface soils may be reduced by contact with mineral salts and organic matter deposited in deep subsoils, the nitrogen reappearing as free ammonia in considerable quantities. Waters of this class may safely contain free ammonia and other nitrogen compounds in quantities which from a chemical view-point would cause suspicion if contained in shallow-well and surface waters.

"The total and relative amounts of the nitrogen compounds in deep-well waters are not, therefore, a quantitative indication of past pollution, as they are in shallow-well waters. The remoteness of the source of these constituents is in itself a security against dangerous contamination.

"A determination of the amount of combined chlorine in waters, particularly surface-waters, is very often valuable in detecting sewage contamination; but in coast countries and in connection with artesian well-waters which may have filtered through soils containing saline matter it loses its significance.

"Total solids are largely derived from the mineral matter in

solution. The degree in which they will affect water for mechanical and manufacturing purposes can be determined by chemical analysis. If contained to the extent of making a water unfit for hygienic purposes the water becomes unpalatable as a rule.

"The analysis of many samples of deep-well waters in England, made under the direction of the 'Rivers Pollution Commission on the Domestic Water Supplies of Great Britain,' lead to the following conclusions, namely: 'Of the different varieties of potable waters, the best for dietetic purposes are spring and deep-well waters; they contain the smallest proportion of organic matter, and are almost always bright, sparkling, palatable, and wholesome, while their uniformity of temperature throughout the year renders them cool and refreshing in summer, and prevents them from freezing in winter. Such waters are of inestimable value to communities, and their conservation and utilization are worthy of the greatest effort of those who have the public health under their charge.' . . . 'In all cases in which spring and deep-well water of good quality are available we recommend that they should be employed in preference to surface- or river-water for domestic supply.'

"The fear of future contamination of the water-supply from increased population and cultivation of the catchment area is very remote, since the prolonged intermittent filtration which the water receives tends to exhaust or to render harmless the organic matter that may have been originally dissolved, while any organic matter which may have been dissolved from the deep subsoils is probably of a vegetable nature and harmless. So far as judgment can be based upon a single chemical determination of the nitrogen compounds, it has not detected pollution in any one of the samples of the local deep-well waters which have been analyzed, when considered from the standpoint of deep-well waters. By comparison with other unpolluted deep-well waters, the local waters certainly show a superiority.

"Of the three local waters, the Tacquard well is shown by the single determination to be the best."

The late Dr. Drown, formerly chemist to the State Board of Health of Massachusetts, furnished a very interesting report

upon the waters of the Galveston wells at a somewhat later date than the report on the preceding pages. It follows in full.

REPORT OF DOCTOR T. M. DROWN, OF BOSTON, MASS.

"MASSACHUSETTS INSTITUTE OF TECHNOLOGY,
BOSTON, MASS., February 6th, 1893.

"*To the Honorable Board of Commissioners of Water-works of the city of Galveston, Texas.*

"Gentlemen: I send you herewith the results of my analyses of the samples of water received from you, bearing on the question of a water-supply for Galveston.

"In addition to the sanitary analysis of the waters, I have made a sufficient number of determinations of the mineral ingredients to indicate their fitness for domestic and industrial uses.

"From the table of analyses it will be seen that the waters from all the Hitchcock wells have a general agreement in composition. They are characterized by rather high contents of alkaline bicarbonates and chlorides, and low contents of lime and magnesia. The waters should be classed, therefore, as 'alkaline saline.'

"I have taken the average of the composition of these five waters as probably representing better the supply obtainable by artesian wells in this locality than the waters from any one well.

"1. With regard to the sanitary properties of the Hitchcock waters, the amount of organic matter in the water, as shown by the albuminoid ammonia and oxygen consumed, is extremely small, and as such is of no significance. The free ammonia is a very common ingredient in deep artesian wells. The origin of it is not always easy to explain, but it certainly has no connection with recent pollution, the only connection which gives it significance in surface-waters and shallow wells. There is no nitrogen present in the form of nitrites or nitrates. The water may be said, therefore, to be perfectly satisfactory as far as its freedom from organic matter or products of decomposition is concerned. I do not think our knowledge of the effects on the human system of mineral matters, when present in very small amounts in drinking-waters, is at present very extensive or accurate. I know of no instance where the continued use

of water of this character has been followed by injurious results. If we deduct the amount of common salt in the average of the five waters from the total solids, we have 34.43 parts of solid matter remaining, or about twenty grains to the gallon. Assuming that these were all sodium carbonate, the amount taken into the system in the regular use of the water would be very small, say seven to eight grains a day. There is no evidence that I know of to indicate that this could prove injurious.

"2. With regard to the technical use of these waters, the advantage of softness in water is one that cannot be overestimated. The expense and annoyance involved in the use of hard waters, both in the household and in industrial works, is so great that a water-supply with little lime and magnesia is much to be prized. The most injurious scale-forming ingredient in boilers is sulphate of lime. This is almost entirely absent in these waters, and the lime and magnesia in the form of carbonates is exceptionally small. As is well known, carbonate of soda is added to hard waters as a preventive of scale in boilers.

"3. With regard to the permanence in the character of these waters on continued pumping I am not able to form an opinion; time alone can decide this question. From the information given by your engineer, Mr. Kiersted, it seems to me very unlikely that the amount of salt will increase by infiltration from the sea. But new underground areas may be drawn upon which contain water of different character. Thus it will be noticed that sample No. 8, from Tacquard's, contains considerable iron, which is absent, or nearly so, in the other samples. When iron is present in well-water in sufficient quantity to cause a precipitation of iron rust, the water becomes unfit for laundry use. The amount of iron in this water (No. 8), 0.0514 part of oxide of iron in 100,000, is very near the limit of precipitation. Again, both the wells of Tacquard's yield water which is more highly colored than the others, and in both the water is slightly harder.

"I do not know whether all these Hitchcock wells have been in continuous use for a sufficient length of time to enable one to say whether the character of the water they now yield is constant. If not, it would seem to be prudent to pump them continuously to their maximum capacity for some weeks to discover if the

Dr. Drown's Sanitary Water Analyses.

In parts per 100,000. Multiply by 0.58 to convert into grains per gallon.

	Residence of W. Tacquard.	Wheeler's well, ¼ mile south of depot, Hitchcock.	Tacquard's, 2¼ miles west of Hitchcock.	Tacquard's, one mile N.W. from Hitchcock.	Hitchcock R. R. well, near depot.	Average of Hitchcock group.	Artesian well four miles north of Alva.	Artesian well at Dickenson.	Artesian well in Houston. Anheuser-Busch Brewing Co.	San Jacinto River at Humble Crossing of Houston E. & W. Texas Railway.
Number of sample	1a*	6	7	8*	9	.....	2	3	4	5
Depth of well	710	600	720	720	720	.....	500	600	840	
Date of examination	Jan. 7, 1893	Jan. 24, 1893	Jan. 24, 1893	Jan. 24, 1893	Jan. 24, 1893	.....	December 30, 1892	Jan. 9, 1893	Jan. 4, 1893	January 4, 1893
Turbidity	None	Slight	Very slight	Slightly milky	Slight	Very slight	None	None	Slightly clayey	Decidedly clayey
Sediment	Very slight	Very slight	Very slight	Slight sa'dy Flocculent	Very slight	Very slight	Very slight	None	Very slight	Considerable earthy
Color	0	0.1	0.2	0.12	0.08	0.1	0	0	0.15	0.55
Odor	None	None	Very faint or none	None	None	None	Decidedly earthy & clayey	None	None	None
Total residue on evaporation	54.20	52.80	56.00	54.00	52.00	53.80	48.00	56.60	27.60	14.40
Albuminoid ammonia (total)	0.0020	0.0038	0.0014	0.0014	0.0022	0.0022	0.0010	0.0020	0.001	0.0238
Free ammonia	0.0230	0.0288	0.0296	0.0214	0.0272	0.0260	0.0256	0.0194	0.0022	0.0024
Nitrites	None	None	None	None	None	None	None	None	None	None
Nitrates	None	None	None	None	None	None	None	None	0.005	None
Oxygen consumed	0.1360	0.1149	0.0365	0.0800	0.0800	0.0895	0.0320	0.0160	None	0.0880
Chlorine	12.1	10.3	14.2	12.1	10.0	11.74	16.40	10.16	2.60	1.20
Hardness by soap solution	3.0	2.6	4.2	3.6	2.6	3.2	0.6	2.2	12.41	6.43
Bicarbonate of lime	3.21	.....	3.75	.....	2.5	3.15				
Bicarbonate of magnesia	1.80	.....	1.92	.....	1.22	1.65				
Sulphate of lime	0.17	.....	0.17	.....	None	0.11	0.27	0.12	2.05	0.17
Iron oxide, FeO	0.0013	None	None	0.0514	None	0.0105	0.0026	0.0039	0.0507	0.0169
Alumina	0.12	.....	0.17	.....	0.09	0.12				
Silica	2.36	.....	2.65	.....	2.48	2.49	1.92	1.54	1.98	5.98
Common salt calculated from chlorine	19.96	16.99	23.43	19.96	16.50	19.37	27.07	16.76	4.29	1.98

* Samples numbered 1a and 8 are from the same well. Sample 1a was carefully collected from the water flowing over the well-casing after the well had been discharging twenty minutes. Sample 8 was collected from a faucet in the residence of Mr. Tacquard when the well was closed off and without permitting it to discharge sufficient water to empty the well-casing and service-pipe.—W. K.

In addition to the foregoing, the analysis of Sample No. 5 from San Jacinto River contains in the original table the following items: Loss on ignition, 3.60. Fixed residue, 10.80. Albuminoid ammonia in solution, 0.0208. Albuminoid ammonia in suspension, 0.0030.

water changes in composition, either in the direction of improvement or deterioration. Any well yielding water high in iron or with iron continuously increasing should be rejected.

"I do not know what system of water-works is under consideration, but the effect of the exposure to light and heat of these waters should be considered if open reservoirs are contemplated. The presence of free ammonia in the waters would afford food for the growth of organisms, which might impart unpleasant tastes and odors to the water.

"With regard to the other samples examined, the Dickenson well is almost identical with the waters from the Hitchcock wells Nos. 1, 6, and 8. The water from Alva is decidedly harder, and therefore much less desirable for general use. The water from Houston is of an entirely different character. It contains much less salt, and its hardness is nearly four times as great as the Hitchcock water.

"The water of San Jacinto River would not furnish an acceptable supply without extensive settling-reservoirs and filter-beds, which, in connection with the expense involved of bringing the water sixty miles, would probably be prohibitive.

"In conclusion I would say that, in my opinion, the waters from Hitchcock, as represented by samples from the residence of Mr. Tacquard, Wheeler's well, and the railroad well, would in their present condition make a satisfactory water-supply for the city of Galveston from a sanitary, domestic, and industrial point of view."

The general superiority of ground-water as a source of public water-supply, hygienically considered, is generally accepted by sanitarians. But how far considerations of quality for hygienic purposes should weigh against considerations of quality for mechanical purposes without prejudice in either direction is somewhat of an open question from other standpoints. As a general proposition, a quality of water which in any particular locality will serve the public and private requirements in a satisfactory degree should prove the most valuable as a commodity, particularly if it be of attractive appearance.

A naturally clear, soft, wholesome water is the ideal water. But water of such a quality is accessible to but comparatively few communities. Usually there is a wide departure of the quality

of available water from that of ideal water in some one or more of the three named particulars, but as a rule the least variation permissible, popularly considered, is in the clearness of the water. A very slight turbidity will arouse a suspicion in the popular mind, even though the wholesomeness of the water may be unimpaired on account of it, and quickly offend the æsthetic taste.

A suspicion of pollution or an offense to the sense of sight becomes all the more intense and aggravated if aroused periodically; that is to say if turbidity is an abnormal condition of a water-supply arising occasionally for a brief interval, a comparison of the condition of the water then with a normally clear water is so striking as to magnify and intensify a suspicion of serious pollution. A community will become accustomed more readily to a water somewhat turbid all of the time than to one similarly affected a part of the time. Thus the turbidity occasionally noticed in ground-water drawn from a shallow-water supply gallery parallel with a river and the accompanying increase of bacteria in the water may be due to too high a rate of filtration through the gravel-bed separating the gallery from the river. This changed condition of the water-supply becomes at once apparent to every consumer, and without knowing the facts the consumers become suspicious of pollution and are quite likely to suspect a direct intake to the river, accidental or otherwise, to meet some contingency of water service—a suspicion affording ground for numerous complaints and unpleasant rumors, outspoken complaint of improper management, all of which are a source of annoyance and often of mutual misunderstandings between the water-works management and its patrons. But in any event a judicious water-works management should desire to avoid either a suspicion of unwholesomeness or an offense to the senses by serving a uniformly clear water, one of so pleasing and attractive appearance that its use as a commodity becomes popular quite as much as a necessity.

There is no desire to be understood as advocating in any measure the clearness of water-supply as a disguise for unwholesomeness. It is mentioned chiefly as a quality which pleases and gives general satisfaction so far as appearances may go, and to this extent should not be disregarded. While there may be little connection

between the qualities of clearness and wholesomeness hygienically considered, still it is known that the provisions which are often necessary to insure clearness often suffice to insure wholesomeness, as, for instance, the complete removal of sediment from a turbid river-water, or the removal of iron from a heavily mineralized ground-water, or microscopic life from a natural lake or reservoir—in fact reflection upon the matter shows that the instances where a clear water is unwholesome are so few as to become notable exceptions wherein individual or community negligence may be a contributing cause.

Many recorded instances of dangerously polluted, though clear, well-water lack substantial confirmation of the source of pollution or the avenue through which the pollution reaches the well. It is believed with a host of observed instances to confirm the belief, that the actual source of well-water pollution is, as has been previously stated, through direct overflow of surface drainage or a fissure underflow affording almost as direct communication with the well as the surface overflow. This is stated as a belief for the reason that no matter how well fortified by observation the statement may be, it must stand in this day and generation simply as a belief until by confirmation of well-directed scientific proof it becomes recorded as a matter of fact. Exceptional and conclusively proven cases of well-water pollution can neither be construed fairly as a reason for wholesale condemnation of well-water supply either for private or public purposes, nor be used sincerely as an illustration to cast suspicion on ground-waters as a class.

It is seldom that we can gratify our desires with regard to the softness of ground-water, as the character of water partakes largely of the geological structure of the ground in the locality where the water-supply is developed. The highly solvent property of water renders it capable of retaining dissolved minerals with a tenacity that requires the exercise of something like the force of chemical action to separate them from the water and to reduce them to a condition in which they can be mechanically removed therefrom. The objection to any process of this kind is to be found in the fact that often it may be merely a substitution of one mineral for another or that the expense involved in

the chemical process is either prohibitive or no less than the expense of repairs resulting from the use of a hard water. Probably the expense is the chief objection to the introduction of most water-softening processes.

Thus far the softening of water is regarded in sort of a relative way and as possessing scarcely a determining influence in the selection of a public water-supply. It will doubtless be given greater consideration in the future as improved and less expensive methods of water-softening are perfected and brought to public notice. Efforts in this direction seem to have been largely confined to the softening of water for industrial use.

It is not intended to pursue in detail this subject of the quality of ground-water, as such a course would be somewhat foreign to the purpose of this treatise and would burden the work with much unnecessary matter. As a rule, ground-water derived from a district suitable for water-supply development is hygienically wholesome when proper precautions are observed to avoid contamination through open channels connecting the underground flow directly with the surface flow or run-off.

Quantity.—The amount of ground-water that is available for water-supply purposes is a perplexing problem to the minds of many a water-works management, as is shown by the many disappointing failures that have attended efforts to secure ground-water supplies. It is believed the problem should not be found so perplexing and uncertain of solution if rationally considered. A few fundamental propositions may perhaps aid in clearing away some of the perplexities. For instance:

No more water can be continuously taken out of the ground than goes into it.

The yield of the ground-water is dependent upon the character and extent of the catchment area and depth of the saturated water-bearing material.

The velocity of flow of ground-water depends upon the character of material through which it must pass in gravitating from a higher to a lower level.

The stability of the ground-water supply depends upon the three considerations above stated as well as upon available ground storage at the point selected for developing the water-supply.

Were these several considerations kept fully in mind when contemplating the development of shallow-ground water there should be little difficulty in approximating correct conclusions. They possess some variations, however, which it is necessary to consider as we proceed.

The amount of water which an artesian basin can furnish permanently is somewhat uncertain because of the difficulty of determining the several governing factors above outlined, and therefore becomes a matter of conjecture except where complete geological surveys have been made, which is a work usually undertaken and carried out only by the general government. It is not unusual to find artesian districts which originally supported flowing wells so overtaxed by the exhaustion of accumulated water storage as to reduce the water pressure and correspondingly the available flow of water to a degree that requires the use of pumping-engines installed at considerable depth below the ground surface to maintain the supply of water. In most instances a reduction of pressure and of flow resulting from constant draft should be anticipated in artesian water-supply development even to the extent of sinking an intake well or installing pumping machinery considerably below the surface of the ground.

Interesting measurements were made by Professor William D. Pence of the artesian flow from wells of the Hitchcock group, near Galveston, Texas, for the purpose of determining the flow of water at different elevations between the surface of the ground and the static level assumed by the water in an extension of the well-casing above the ground.

The flow of the well ceased at a height of about 30 feet above the surface of the ground.

In the following tabulations the heights represent the distance from the top of the $7\frac{1}{2}$-inch well-casing near the surface of the ground upward to the several centers of discharge in a 3-inch rising pipe tapped into a cap screwed onto the well-casing.

DISCHARGE OF ARTESIAN WELL AT HITCHCOCK, TEXAS, MAY 19, 1891. MEASUREMENTS TAKEN WITH TWO-INCH EMPIRE METER.

Height above top of well-casing.	Discharge in gallons per 24 hours.
28.00 feet	Scarcely any
25.35 "	8,022
20.45 "	32,089
15.55 "	40,000
10.62 "	51,343
5.26 "	70,551
0.76 "	94,966

DISCHARGE OF ARTESIAN WELL AT HITCHCOCK, TEXAS, JUNE 5, 1891. MEASUREMENTS TAKEN WITH FOUR-INCH CROWN METER.

Height above top of well-casing.	Discharge in gallons per 24 hours
28.00 feet	Scarcely any
20.4 "	28,245
15.21 "	48,470
10.63 "	67,320
5.39 "	84,374
1.03 "	98,616

It will be observed that the discharge varies almost directly with the pressure or head and that for each one foot of head or equivalent pressure below the level which the water naturally assumes in the stand-pipe connected with the well-casing there is an equivalent acceleration of discharge of about 3400 gallons per twenty-four hours; for instance, if the discharge from an orifice in the stand-pipe one foot below the static level of water therein is 3400 gallons, then the discharge 50 feet below said level should be 170,000 gallons per day or 340,000 gallons at a level 100 feet below said level.

The experience with artesian wells at Memphis indicates a discharge of about 1,000,000 gallons per day from fifty wells for each 4 feet of draft, or a rate of increase of about 5000 gallons per twenty-four hours per well for each one foot of draft.

It is estimated that the available flow from a single pumping-station of the Memphis district is about 25,000,000 gallons per day under a draft about 60 feet below static water-level. In the Hitchcock district, where the available draft is about 50 feet, the station yield should approximate about 15,000,000 gallons per twenty-four hours. But the water-supply development need not be confined to a single pumping-station and its group of wells, as in an extensive artesian district auxiliary stations and other groups of wells may be located beyond the zone of influence of the first group—possibly several miles therefrom as local conditions suggest—and operated by power transmitted from the original or central station.

Artesian water-supplies, particularly those derived from sand or sand rock, are exceedingly desirable and should be developed to the highest possible degree and with the greatest amount of care. They can scarcely be equalled by any artificially filtered surface-water supply, considered hygienically.

Shallow ground-water supplies are usually developed from sand and gravel deposits in the valleys of creeks and rivers. In a valley where the stream flows over a rock bed the sand deposits are thin and usually so exhaustively drained as to afford but a very small yield of water. The attempts to develop a public water-supply in valleys of this character have failed almost without exception, for there is no permanent water storage of any great volume.

Valleys like those of the important rivers of the central states often possess immense deposits of sand and gravel extending 50 or more feet below the river-beds. These deposits afford immense reservoirs for the permanent storage of water, which can be drawn upon for large volumes of water year after year with no fear of depletion. The valleys of the Mississippi and Missouri Rivers afford a striking example of sand deposits of this kind, but, strange as it may seem, the value of these deposits as sources of water-supply has as yet never received in practice the public appreciation they deserve. The impression seems to prevail in communities located along these rivers that the rivers

themselves are the best available source of supply, and immense sums of money have been expended in the erection of water purification works without a thought apparently of considering the availability of a clear and wholesome supply from the sand beds beneath the rivers.

Any impression that the deposits are generally a mixture of alluvia and quicksand, materials which are known to be altogether too fine for the free flow of water, is erroneous. While deposits of this character may be encountered above the level of the river-bed, it is generally true that the deeper deposits below the bed of the river are composed very largely of clean coarse sand intermingled with gravel. Frequently the coarse sand is in strata separated by thin layers of tenacious clay. In some districts along the Mississippi River the deeper deposits for 20 to 30 feet are composed largely of heavy gravel and sand, with scarcely a suggestion of the presence of quicksand. The heavy and coarse deposits can be made to yield a large supply of water by proper methods of development.

In estimating the available yield of water from sand deposits of these and similar river valleys the matter of catchment area is scarcely to be taken into consideration outside the valley proper, for the river is an important factor in keeping the sand deposits supplied with water should they be drawn upon. The situation in this regard can probably be more clearly presented by referring to tests made for the purpose of developing the water-supply conditions in the valley of each of the two rivers named.

In 1902 a study was commenced of the ground-water-supply conditions of the Mississippi River valley at a point about two and one-half miles below the city of Muscatine, Iowa, having in view the new water-supply works for that city. The studies finally resulted in a decision to abandon the Mississippi River, the source of supply for twenty-five years of the original water-works, and to develop ground-water at the site of tests referred to. They also developed much interesting information regarding ground-water conditions.

Fig. 1 is a diagram of a series of test wells excavated to bed-rock or clay on a line about right angles with the Mississippi River.

Each well after reaching the rock or clay substratum was withdrawn a few feet to admit of free percolation of water into it, except well 1, which had a strainer about 8 feet long composed of three rings of vertical slots, nine slots to each ring, $\frac{1}{4}$ to $\frac{3}{8}$ of an inch wide and respectively $21\frac{1}{2}$, $24\frac{1}{2}$, and $33\frac{1}{2}$ inches long, each ring of slots being separated by 8 or 10 inches of solid metal. The combined area of the slots was about four and one-half times the sectional area of the 8-inch well-casing. The strainer is the design of William Molis, superintendent of water-works, Muscatine, Iowa.

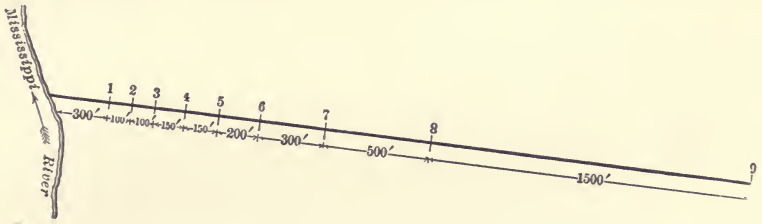


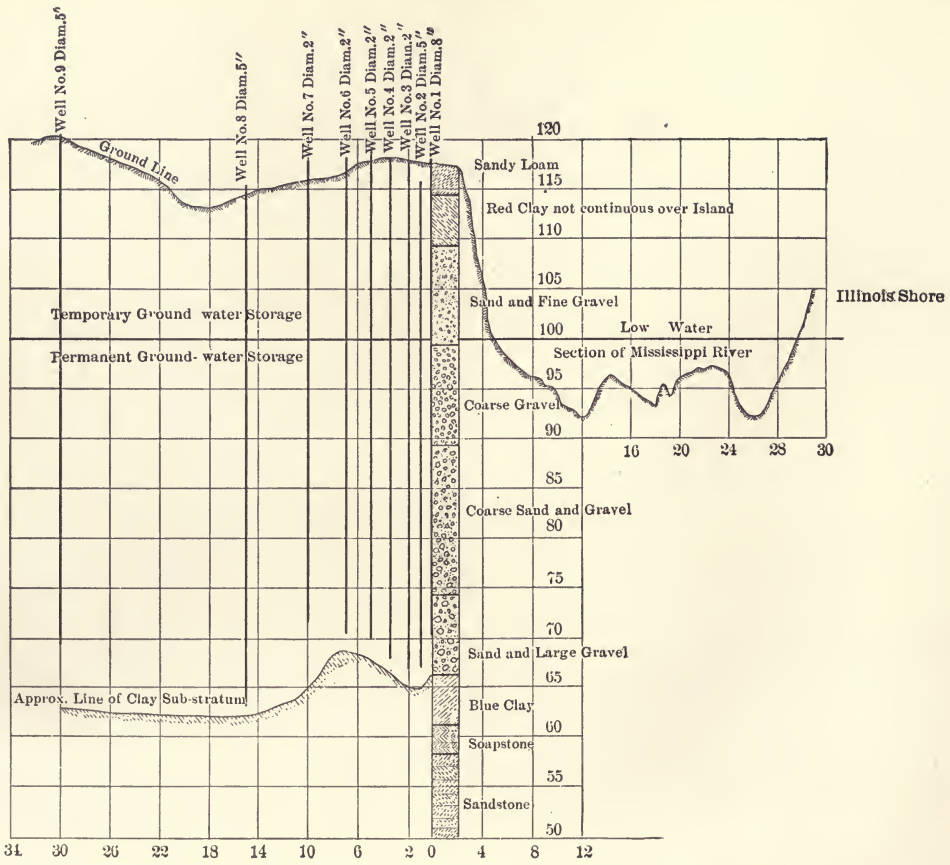
FIG. 1.

Muscatine Island (island in name only), where the test was conducted, is several miles wide, with a slightly undulating surface. A high-water slough skirts the bluffs on the western side and the Mississippi River washes the eastern side of the island. The rainfall upon the island, which is not evaporated or absorbed by vegetation, passes almost entirely into the underflow; scarcely any of it disappears in surface run-off.

This land is under cultivation for the raising of garden truck and accordingly extensively fertilized. Storm-water rapidly disappears, showing thorough underdrainage.

Plate I gives an idea of the geological structure of the island, at least for 3000 feet inland, and shows a vertical section along the line of test-wells and soundings of the Mississippi River to the Illinois shore. It also serves to show the immense permanent storage capacity of the gravel deposits below the low-water stage of the river, and the additional temporary storage between the low-water and the usual high-water marks. The tangible value of even the temporary storage is not to be underestimated along such rivers as the Mississippi and Missouri, for as we write the Mississippi River has been 5 feet or more above low water for nearly a year,

PLATE I.



Geological Structure of Muscatine Island.

and accordingly has served to maintain a high ground-water level. However the temporary storage is not an element that can be depended upon for water-supply year in and year out, for during the period between 1878 and 1902 there were ten years when the average gauge-reading of the Mississippi River at Muscatine was below the 5-foot stage and two years when the stage of the river was below the 5-foot mark for the entire year.

The problems which were presented for solution in the Muscatine study of ground-water supply are the same as those presented in other similar localities, and the methods there pursued are characteristic of those which may be pursued elsewhere with such variations as may suit local requirements. In general the essential problems for solution are as follows:

1. What is the extent and available depth of the permanent ground-water storage?

2. What is the available velocity of flow through the deposits of sand and gravel underlying the river valley and the probable greatest permissible hydraulic slope?

3. What is the source from which the gravel-beds are supplied with water and how much of the water thus supplied is available for use?

4. What is the chemical and bacteriological condition of the ground-water?

5. Is there danger of contamination of the underflow from drainage of the city above or from farms in the immediate vicinity of the site of the tests and observations?

6. What extent of water-supply development work is required for the desired supply of water?

The borings were made for only a distance of 3000 feet from the river, affording information of a practically uniform geological formation of the lower strata of water-bearing material. Private wells for a radius of a mile or more from the site of the tests, equipped with centrifugal pumps, showed the water-bearing material to continue much beyond the range of the test-wells. So far as could be ascertained the bed of the Mississippi River is of the same formation, and it was assumed the gravel formation extended even beyond the Illinois side of the river. Therefore without pursuing investigations to actually determine the full

extent of the gravel deposits, a work evidently involving large expense, reasonable assurance was developed to indicate at least two square miles of available catchment area which should absorb sufficient rainfall, if rainfall only could be relied upon to support the gravel-beds, to yield at least 454,000 gallons of water per square mile of catchment area, or an average of 813,500 gallons per twenty-four hours, the smaller computation being based upon $9\frac{1}{2}$ inches or 35 per cent of the minimum annual rainfall, and the larger amount being based upon 15 inches or 45 per cent of the average rainfall.

The local conditions seemed to require a basement level of the pump-pits at about 6 feet above low water, and the pump-plunger about 9 or 10 feet above low water, and a suction draft to a level 10 feet below low water or possibly a little more.

With due allowance for hydraulic slope of the ground-water the available permanent storage which can be drawn upon in addition to the rainfall yield is computed at 130,000,000 gallons per square mile.

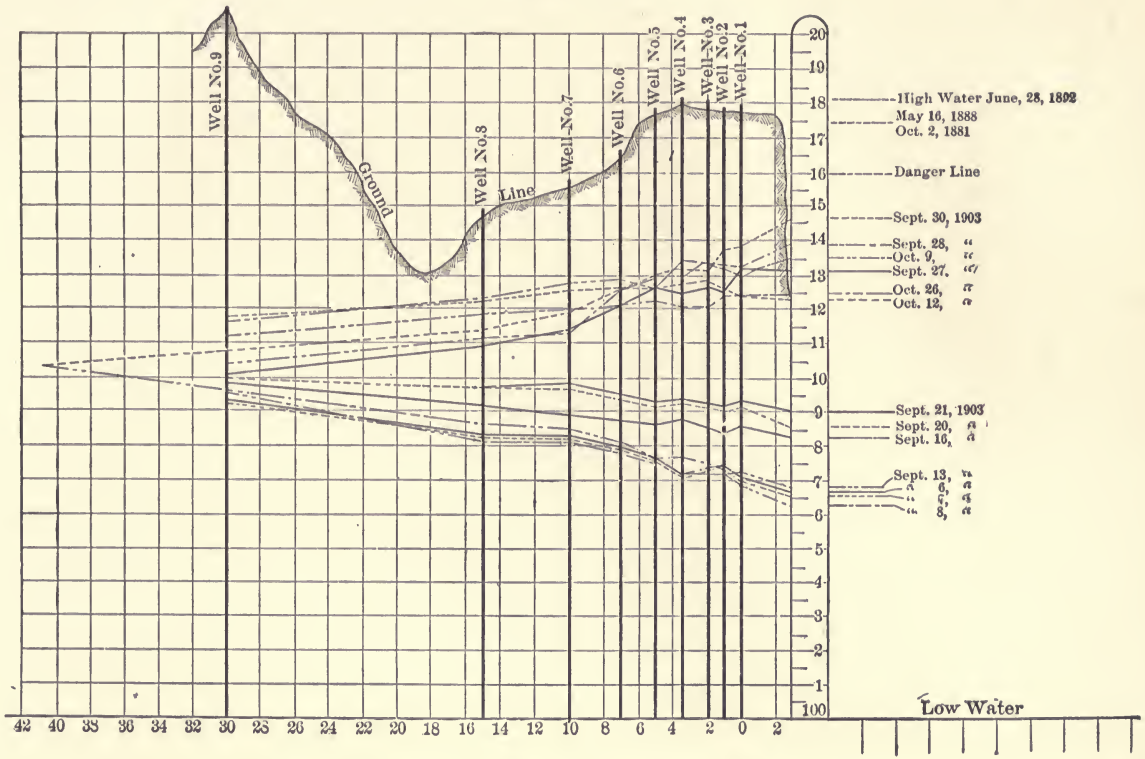
During the months of September and October, 1902, practically simultaneous observations were made of the level of the water in the nine test-wells and of the river. The stage of the river was practically constant for the month preceding the beginning of the observations. The period covered by the observations embrace a period of rather rapid rise of the river and a corresponding fluctuation of the ground-water level in all of the test-wells.

The fourteen representative series of water-level observations are represented in the diagram, Plate II. Other observations were made between September 21 and 26, in connection with a pumping test, which are the subject of special comment further on.

The four water-surface lines of September 6 to 13, inclusive, show a pronounced slope of the ground-water table towards the river of an average of 0.8 of a foot in 1000 feet and necessarily a movement of the absorbed rainfall from remote points of the island towards the river.

A rise of 2 feet in the river between September 13 and 21 produced a corresponding but gradually decreasing rise inland,

PLATE II.



Water-level Observation, Muscatine, Iowa.

even further inland than well 9, where the recorded rise was 0.3 of a foot. The average slope of the ground-water table between the dates named is about 6 inches per 1000 feet. It is observed that during this interval of progressive rise the slope remains downward towards the river, showing a persistency of the ground-water to advance in that direction notwithstanding the check imposed by the rise of the river. This check or damming of the ground-water at the river-front compelled the advancing underflow to fill the voids of the sand in the wedge between the water planes of September 13 and 21. In order to accomplish the filling of this wedge 2 feet in height at the river-front and with an apex about 3300 feet inland, the water in the saturated gravel of the island down to the clay substratum, a depth of about 43.5 feet, must have advanced laterally at least 58 feet through the gravel or at the rate of 7.25 feet per day, or nearly the equivalent of $2\frac{1}{2}$ feet of solid water 43.5 feet high after making correction for a rainfall of $1\frac{1}{2}$ inches during the interval.

As the river continued to rise the slope of the ground-water was reversed to an inclination from the river inland about September 23, and continued reversed until after the summit of the rise was reached on September 30. The average inclination of the slope inland during this period was 1.13 feet per 1000 feet. The advance of river-water inland to fill the voids in the sand between the curves of the dates named is computed in a manner similar to the above computation to have been 41.1 or at the rate of 13.7 feet per day, equivalent to 4.6 feet of solid water 44 feet high. In the foregoing computation a void space of one-quarter the mass was allowed for the wedge embraced between the several ground-water planes in order to allow for moisture permanently retained by capillary attraction, and one-third the mass was considered as available void space in computing lateral sectional flow through saturated material.

The matter may be put into convenient shape for computation by means of the Darcy formula, namely,

$$v = ki,$$

wherein v = velocity in feet per day;
i = inclination of ground-water surface in a given distance divided by that distance;
k = a factor depending upon the porosity of material through which the underflow passes.

Interval.	v. Velocity in feet per day of solid water.	i. Inclination.	k. Factor.
September 13 to 21.....	2.5	0.0005	5000
" 20 to 21.....	1.55	0.000272	5700
" 27 to 30.....	4.63	0.00113	4100

Average value of k is 4900 in round numbers.

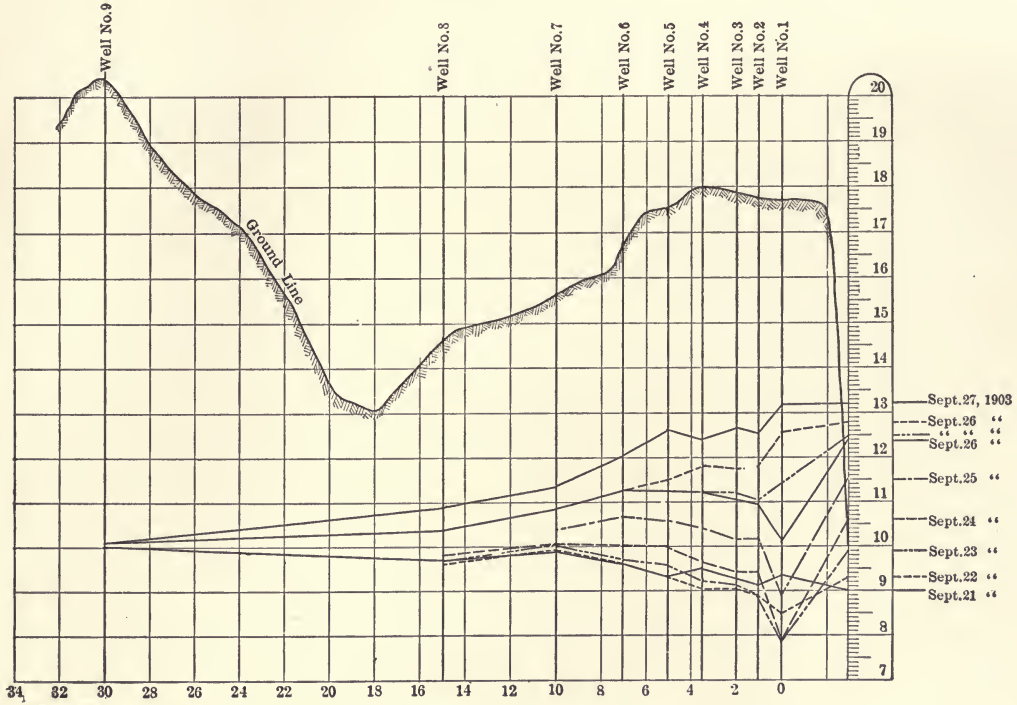
In order to obtain the actual advance as measured in the voids of the sand the tabulated velocities should be multiplied by 3 and the corresponding average value of k would then be 14,700.

During the interval between the dates September 21 and 27 the condition of the ground-water table was observed in the several wells while pumping on one of the wells was in progress. Plate III contains a diagram showing the fluctuations referred to.

A centrifugal pump was connected directly with the 8-inch casing of No. 1 well, the details of which well are described on a previous page, and operated practically continuously by means of a belted connection with a traction-engine, from 7 A.M. of the 21st to 12 P.M. of the 26th, one hundred and twenty-five and one-half hours. The object of this test was for the purpose of deriving information regarding the porosity of the gravel deposits. Accordingly the actual amount of water pumped from the well was regarded as purely an incidental matter and of itself capable of furnishing little or no reliable information of the kind desired. Such approximate measurements as were made of the discharge of the pump as the water passed through a wooden flume to the river indicate a discharge at the rate of 1,500,000 gallons per twenty-four hours.

An interesting fact connected with the test is the one that the river rose over 3 feet between the morning of the 21st and the 26th and that a corresponding rise was observed in most

PLATE III.



Ground-water Table during Pump Test, Muscatine, Iowa.

of the wells notwithstanding the large amount of water that was abstracted from well No. 1.

Plate III shows the zone affected by the pumping to extend about 700 feet from the pump well and to present a face about 1400 feet in diameter to the river-front. The depth to the clay substratum below the water-table at this well is about 45 feet.

When pumping ceased at 12.25 P.M. the zone of depleted sand as the result of pumping was about 1000 feet in diameter. Between that hour and 4.30 P.M. of the same day the inflow from the river so filled the depleted sand that there was a continuous slope to test-well 8 and almost a level water-table for the next 1500 feet inland, or to test-well 9. On the morning of September 27 the influence of the rising water-table caused by inflow of water from the river became decidedly apparent at and beyond test-well 8, and were it not for the heavy pumping of the preceding days the water-table would doubtless have presented a more uniform slope. As soon as pumping ceased the inflow of water was comparatively heavy and must have been influenced very largely by the steep slope which then obtained between the river-front and pump well No. 1 even as late as the morning of the 27th. The average slope between 12.25 P.M. of September 26 and 7 A.M. of the 27th was 2.8 feet per 1000 feet, and the estimated volume of the cone of depleted sand of a base 1000 feet in diameter which was filled with water between 12.25 and 4.30 P.M. and of the wedge between the water-table of the last stated hour of the 26th and that of the morning of the 27th divided by 48,000 square feet of river frontage gives approximately a velocity of flow through the sand of 45 feet per twenty-four hours or about 15 feet of solid water. From these data we compute by the formula $v = ki$ the value of the coefficient of porosity (k) to be 5360. A previous method of computation gave the value of k at 6200, but it is believed the later computation fits the observations more correctly. In view of these computations a factor of porosity 4000 to 5000 appears to suit the Muscatine conditions and admits of the following local rule, namely:

Multiply the observed slope of the ground-water table expressed in feet per 1000 feet by the factor 4 to 5 to get velocity of underflow

expressed as solid water, or 12 to 15 to get velocity expressed as lateral movement through sand, assuming the voids in sand to be one-third the mass.

No mechanical analysis was made of the Muscatine gravels and sands at the time of the tests referred to, but later when excavating for a permanent water-supply well, several samples of the sand and gravel penetrated by the boring were collected and analyzed, with the following result.

MECHANICAL ANALYSES OF MUSCATINE SAND AND GRAVEL.

Number of sieve.	Per cent of sand finer than stated sieve numbers.			
	1.	2.	3.	4.
Rejected by 6....	5.5	18.5	54.5	36.8
6	94.2	80.50	45 5	63.2
10.....	83.1	58.08	27.2	25.8
14.....	63.3	41 0	13.1	7.6
18.....	44.0	32.4	8.4	3.0
20.....	31.5	24.1	6.2	1.7
40.....	1.1	2.3	2 0	0.2
60.....	0.09	0 35	1.1	0.02
80.....	0.00	0.00	0.2	

In order to form some conception of the classification of the Muscatine sand we have deduced the value of the coefficient of porosity (k) of assorted and washed sands of various sizes from the tabulated results of experiments contained on pages 210, 211, and 244 of Professor F. H. King's report in the Nineteenth Annual Report of the Geological Survey.

The factors in the following table indicate that the effective size of the Muscatine sand should be about the grade of a coarse concrete sand, the nearest tabulated size of sand being 0.7146 millimeter, which is little less than one thirty-second of an inch or somewhere near the size of sand which will pass a sieve of 20 meshes to the lineal inch and will be retained by one of 40 meshes to the lineal inch. However, the analysis of the Muscatine sand indicates a coarser grade of sand, at least samples 3 and 4, than that indicated by the table, a circumstance due no doubt to the fact that the sand experimented with by Professor King was of uniform sized grains, possessing a uniformity coefficient of unity, while the Muscatine sand is an unassorted sand of various sized

grains. It should be recollected that the velocities stated in the following table are measured as volume of solid water passing in a unit of time, and that for a lateral velocity in the voids of the sand the tabulated velocities should be multiplied by about 3.

FLOW OF WATER THROUGH SANDS AND SANDSTONES IN CUBIC FEET PER MINUTE PER SQUARE FOOT OF SECTION UNDER A PRESSURE GRADIENT OF 1 IN 10.

Series Number of sand.	Size of sand grains in millimeter.	Flow in cubic feet per minute.	* Velocity in feet per 24 hours. Grad. 1 in 10.	* Value of coefficient (k).
No. 8 quartz.	2.54	5.2268	7526.6	75266
" 7 "	1.808	3.649	5254.6	52546
" 6 "	1.451	1.8481	2161.3	21613
" 5½ "	1.217	1.3582	1955.8	19558
" 5 "	1.095	1.2232	1761.4	17614
" 4 "	0.9149	0.8242	1186.8	11868
" 3 "	0.7988	0.5084	732.1	7321
" 2 "	0.7146	0.3295	470.9	4709
" 1 "	0.6006	0.2321	334.2	3342
" 0 "	0.5169	0.1767	254.4	2544
" 100 "	0.1018	0.0041	5.9	59
Dunville sandstone.	0.0363	0.029	0.3	3
Madison sandstone.	0.0818	0.246	2.5	25

* Author's Computation.

In this connection it is interesting to know what the coefficient of porosity may be of natural sands and gravels. Apropos of this part of our subject are the experiments of the Metropolitan Water and Sewerage Board of Boston, Massachusetts, conducted to determine the rate of percolation through sand and soils, summarized by F. P. Stearns, chief engineer of that Board, as follows:

"Amount of filtration in gallons per day through an area of 10,000 sq. ft. of different materials, with the loss of head of 1 foot in 10 feet.

"Coarse	sand,	the	average	of	three	experiments	2,200,000
Medium	"	"	"	"	six	"	400,000
Fine	"	"	"	"	two	"	90,000
Very fine	"	"	"	"	"	"	7,200
Soil		"	"	"	"	"	510"

The mechanical analysis of the several grades of material above described is not given in the article referred to, but the data are sufficient for the computation of the velocity stated in terms of solid water and the coefficient of porosity as follows:

	Velocity.	Coefficient (k).
Coarse sand	29.3	293.0
Medium sand	5.3	53.0
Fine sand	1.2	12.0
Very fine sand	0.96	9.6
Soil	0.0068	0.068

It is seen that the grade of sand has much to do with the permissible velocity of water flowing through it; that is to say, the coarser the sand and the more uniform the size of the individual grains of sand the more freely does water flow through it. As a rule, however, a sand deposit is a heterogeneous mixture of intermingled large- and small-sized grains. If the small grains are very fine and constitute a considerable percentage of the total volume, the voids of the deposit may be so thoroughly filled as to admit of but slow percolation, notwithstanding the presence of even a considerable amount of coarse gravel. Instances of this kind are not uncommon.

A striking example of this kind is to be found in the following mechanical analysis of a sand from a district bordering on the Ohio River.

Sand	rejected	by	sieve	No.	6	36	per cent.
"	passed	"	"	"	6	64	" "
"	"	"	"	"	10	57	" "
"	"	"	"	"	14	53.2	" "
"	"	"	"	"	18	51.2	" "
"	"	"	"	"	20	49.2	" "
"	"	"	"	"	40	34.7	" "
"	"	"	"	"	60	20.0	" "
"	"	"	"	"	80	16.5	" "
"	"	"	"	"	100	12.7	" "

The 12.7 per cent of the sand which passed the No. 100 sieve contained a considerable amount of silt which by washing and reweighing was found to be about 2½ per cent.

The uniformity coefficient is about 12, indicating a heterogeneous mixture of gravel stones of considerable size with a highly assorted sand in which there is a large percentage of fine grains.

The result of Allen Hazen's investigations under the direction of the Board of Health of Massachussetts indicates that the finer 10 per cent. of the sand grains controls the percolating capacity

of sand and gravel, and suggests expressing sand texture in terms of a "uniformity coefficient," found by dividing the size of the sand grain separating the coarser 40 per cent from the finer 60 per cent by the size of the grain separating the finer 10 per cent from the coarser 90 per cent.

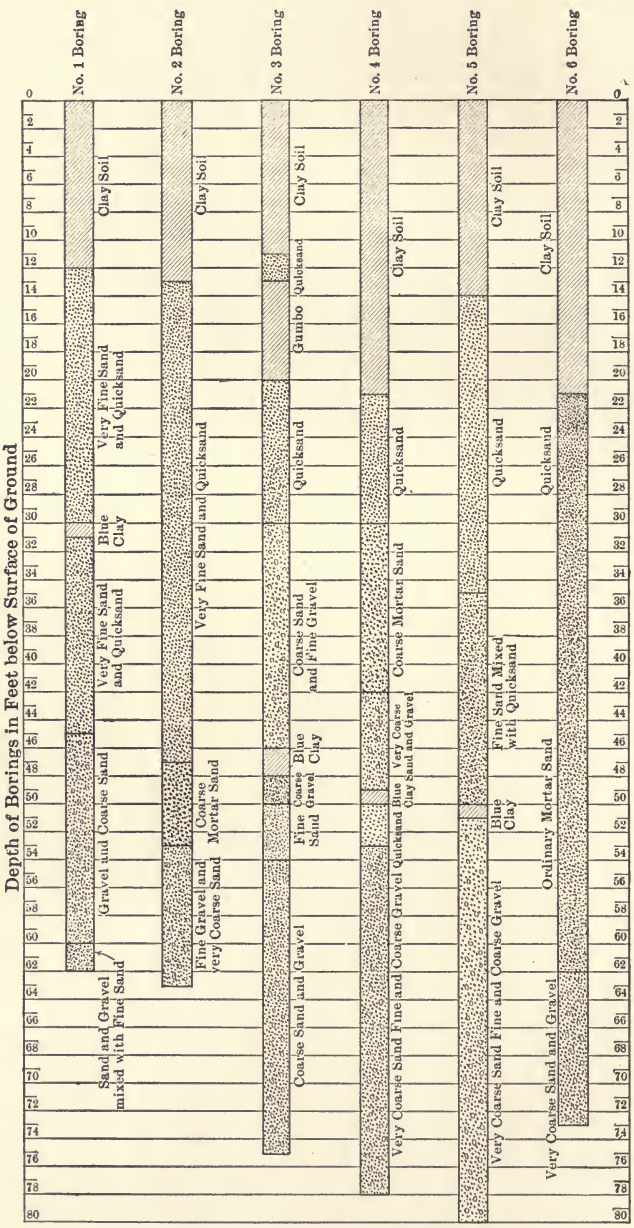
It seems, however, as though the entire relation of sand texture might be more conveniently expressed by a nomenclature referring to a series of graduated sieves and the uniformity coefficient expressed as the ratio between the number of meshes to the lineal inch of the sieve separating the coarser 90 per cent from the finer 10 per cent and the number of meshes to the lineal inch of the sieve separating the finer 60 per cent from the coarser 40 per cent. The author suggests a nest of sieves for the purpose of analysis graduated 100, 80, 60, 50, 40, 30, 20, 18, 10, and 6 inches respectively to the lineal inch. The size of the wire entering into the construction of the sieves must necessarily vary in gauge and should be fixed in accordance with some practical standard. Having once arrived at a standard nest of sieves the sand can be classified as No. so and so, corresponding to the sieve which allows it to pass after a definite period of agitation of the sieves nested. Thus a sand which passes a No. 6 sieve and is retained by a No. 10 sieve, or the next size finer whatever it may be, would be classified as No. 6, etc.

To illustrate the method reference is made to a table representing a mechanical analysis of sand from a series of borings in the valley of the Misssouri River shown graphically on Plate IV.

ANALYSES OF MISSOURI RIVER SANDS.

No. of sieve.	Per cent of sand finer than stated sieve number.					
	Well No. 1.	Well No. 2.	Well No. 5.	Well No. 6.	Missouri River channel.	Quicksand.
Rejected. .	3.07	9.34	20.82	0.0	5.94	
6.....	96.93	90.66	79.18	100.00	94.06	
10.....	91.93	80.63	64.05	99.42	87.25	
14.....	85.97	72.24	49.14	98.82	80.13	
18.....	80.23	64.70	37.89	98.20	73.48	
20.....	73.12	55.70	27.10	97.07	65.95	100.00
40.....	24.12	17.32	3.87	75.27	29.07	96.17
60.....	6.87	7.81	1.26	17.81	12.07	85.46
80.....	4.01	5.60	0.91	9.34	8.44	65.54
100.....	2.16	3.52	0.68	4.94	5.69	40.92

PLATE IV.



Borings along the Missouri River near Fort Leavenworth.

Upon platting the percentages corresponding to the above-stated sieve sizes and connecting the consecutive points so platted the sieve number may be read off corresponding to the effective size of the sand, or that which separates the finer 10 per cent from the coarser 90 per cent, also the sieve number corresponding to the separation of the finer 60 per cent from the coarser 40 per cent. Dividing the former by the latter gives the uniformity coefficient. The following table expresses the results of the platting of the preceding table.

No. of test-well.	Sieve corresponding to the separation of		Uniformity coefficient.	Approximate effective size in millimeter.
	Finer 10% from the coarser 90%.	Finer 60% from the coarser 40%.		
1.....	54.5	25.2	2.12	0.36
2.....	56.0	19.0	2.95	0.35
5.....	34.5	11.5	3.00	0.55
6.....	79.0	45.5	1.74	0.21
Missouri River. . . .	72.0	23.5	3.07	0.29

In order to show the relation between the sieve number and the effective size of the sand grain, the tabulated results of Allen Hazen's experiments in the mechanical analysis of sands contained in the Twenty-fourth Report of the State Board of Health of Massachusetts have been platted, and from the diagram so constructed there has been tabulated the relation above referred to over as wide a range as it is necessary to consider in practice. The table follows.

TABLE SHOWING THE RELATION BETWEEN THE SIEVE NUMBER AND THE EFFECTIVE SIZE OF SAND GRAINS.

Sieve No.	Effective size in millimeter.	Sieve No.	Effective size in millimeter.
140	0.135	40	0.46
120	0.155	30	0.71
100	0.18	20	0.96
90	0.20	18	1.10
80	0.22	14	1.52
70	0.24	10	2.04
60	0.32	6	3.90
50	0.39		

The essential advantage to be derived in a classification of sand by the sieve number is that that nomenclature is more generally understood and is in close association with the only apparatus that is usually available in field operations of the mechanical analysis of sand.

The method may be carried a step farther and a series of factors deduced by which the measured slope of the ground-water table may be multiplied to give the velocity of underflow and furthermore these factors may be tabulated in conjunction with corresponding sieve numbers in a manner to facilitate the work of computation.

There has been already given a series of such factors (k) in formula $v=ki$ in the table expressing the results of Professor King's experiments on page 31. However, these factors representing the condition of minimum resistance offered by sand to the flow of water are not generally applicable, for the reason that they represent a measure of the resistance of sorted sands of uniform grain possessing a uniformity coefficient of unity which is a condition of material not encountered in field operations.

Allen Hazen gives a formula for the flow of water through unassorted sands as follows:

$$v=cd^2\frac{h}{l}\left(\frac{t\text{ Fahr.}+10}{60}\right),$$

where v is the velocity of water in meters daily in a solid column of water of the same area as that of the sand,

c is a constant factor which present experiments indicate to be approximately 1000,

d is the effective size of sand gains in millimeters,

h is the loss of head,

l is the thickness of sand through which water passes,

t is the temperature (Fahr.).

Assume a temperature t of 50 degrees, which is approximately the temperature of ground-water, and that the entire head or pressure of water for a depth equal to the thickness of sand is consumed in producing velocity, as would be the case of ver-

tical flow with unobstructed discharge in a layer of sand saturated to the top, then Hazen's formula reduces to the simple expression $v = cd^2$, the factors cd^2 being equivalent to k for vertical flow in the Darcy formula previously described.

Upon using the values of effective size for which the corresponding sieve numbers are known, as given on page 35, and interpolating for intermediate sieve numbers, we have the results expressed in the following table.

VALUES OF COEFFICIENT OF RESISTANCE k IN FORMULA $v = ki$, CORRESPONDING TO STATED SIEVE NUMBERS.

No. of sieve.	k	No. of sieve.	k	No. of sieve.	k
6	49889	24	2474	60	336
8	31770	26	2200	70	189
10	13650	28	1926	80	159
12	10614	30	1653	90	131
14	7578	32	1461	100	106
16	5773	34	1269	120	79
18	3969	36	1077	140	60
20	3022	40	694	200	40
22	2748	50	499		

EXAMPLE.—The velocity of flow through a sand 90 per cent of which by weight is retained by a No. 60 sieve when the ground-water slope is 1 foot in 1000 feet is $v = 336 \times 1/1000 = 0.336$ feet per day of solid water or 1 foot per day in terms of the voids in the sand.

EXAMPLE.—What is the grade of the sand which with a slope of 1 foot in 100 feet admits of a velocity of 10 feet per day in terms of void space? In this case $\frac{10}{3} = k\frac{1}{100}$ or $k = 333$, corresponding to a number 60 sand in preceding table.

A recent experiment to determine the flow of water through a No. 55 sand resulted in a measured vertical velocity of flow of 511 feet of a solid column of water when the water covered the sand to a depth of $14\frac{1}{2}$ inches and a corresponding value of k of 425 for sand just covered with water, the uniformity coefficient being 2.1.

Upon platting the results of the analysis of the Muscatine sand as tabulated on page 30 the following table is derived.

EFFECTIVE SIZE AND UNIFORMITY COEFFICIENT OF MUSCATINE SANDS.

Sample.	Sieve corresponding to the separation of		Uniformity coefficient.	Approximate effective size in millimeters.
	Finer 10% from coarser 90%.	Finer 60% from coarser 40%.		
1	32.0	15.0	2.1	0.81
2	33.0	9.5	3.5	0.86
3	17.0			1.20
4	13.5	6.25	2.0	1.59

Now it will be remembered that the value of k in the formula $v=ki$ as found by computation based upon water-level measurements was 4900 for velocity in terms of a solid column of water. The same factor k computed from the table of coefficients on page 37 for the sieve numbers 32, 33, 17, and 13.5 is respectively 1461, 1365, 4871, and 8337, or an average of 4008 without regard to the respective depths of the several grades of water-bearing sand, which information just at present is not available, but which when definitely known will doubtless modify somewhat this value of k.

Serviceable water-bearing sands are seldom finer than No. 70, and usually of a range of 60 to 30. Deep beds of water-bearing gravel and sand like the Muscatine deposits are rather exceptional.

The water-bearing capacity of a sand is an essential consideration in judging of its value for water-supply purposes, for unless it is of a grade which will admit of a free circulation of water the results of water-supply development may be very disappointing. Wherever the natural velocity of underflow and the slope of water-table can be measured with a reasonable degree of approximation, it is best to take these measurements, as they offer direct evidence of the flow through the sand deposits. But such measurements are not always feasible and usually some substitute must be accepted. The best substitute is that of a careful mechanical analysis and a survey of the sand deposit of sufficient extent to admit of capacity determination of the character herein described.

Direct pumping from a well seldom furnishes reliable evidence of the permanent supplying capacity of sand deposits

unless continued for a long period of time, much longer in fact than is afforded most investigators of water-supplies. The results determined by pumping are usually in excess of the permanent yield by the amount of water contributed to the well from local storage. For instance, if one were to attempt to determine the flow of water into a pond by pumping from the pond, it is evident that so long as the level of the water was depressed as the result of pumping, a draft is made upon storage and the volume of water delivered by the pumps is in excess of the actual inflow and will continue to be in excess until the level of the pond ceases to fall or the storage is completely exhausted. The same principle holds true in pumping from a well. The storage in the sand about the well augments the yield of the well and will continue to do so until exhausted and a stable water-table is established in the sand deposit. However a long period of time is sometimes required to establish a stable water-table for the reason that available storage extends a considerable distance from the well and gives up its water in a progressively decreasing volume moving at a low velocity.

A short-time pumping test furnishes direct information of value when observations are made of the curve assumed by the water in the ground approaching the well. A steep or abrupt pitch downward toward the water surface in the well over a comparatively restricted circumscribing area indicates a fine sand of high resistance, while a flatter pitch covering a more extensive area affords information of the presence of a coarse sand of comparatively low resistance. It is apparent therefore that for a given rate of yield a fine sand deposit must be much deeper theoretically than a similar deposit of coarse sand.

There are records of cases where the depletion of ground storage as the result of continuous draft has been so gradual as to extend over a period of several years, showing conclusively the erroneous conception which usually arises from the results of short-period pumping tests.

A tunnel driven under a mountain will often open subterranean fissures from which water flows into the tunnel in great volumes for a considerable period of time and finally dwindles

down to a comparatively insignificant flow. Other instances of a similar character are encountered when shafts have been sunk into water bearing sand rock and tunnels driven laterally for a considerable distance. At first water flows abundantly, but as the draft continues and the water pressure falls as the natural storage is depleted, the flow diminishes and finally must be reduced to the annual rainfall supply. Similar experience has followed the use of filter galleries in sand and gravel deposits—all going to show that a study of the mechanical make-up, depth and extent of water-bearing deposits, as well as available ground-water slope, constitute essential elements of study in formulating correct conclusions with regard to the probable permanent yielding capacity. These observations have their exceptions, as, for instance, when water is taken from a well penetrating the permanent storage deposits underlying such rivers as the Missouri and Mississippi, hereinbefore alluded to; for then the influence of the river has a pronounced effect in restoring rapidly the depleted storage resulting from a heavy draft upon ground-water. The Muscatine studies afford a striking example of river influence as shown by the diagram representing the result of these studies. It is seen that under normal conditions there is a pronounced slope and flow of ground-water towards the river, and it is obvious that the inclination of the ground-water table riverward must be modified both by a departure of rainfall from the normal and by river fluctuation, with a tendency to flatten during a drought and a prolonged stable stage of the river. It is also clear that a rapid rise of the river dams the ground-water, checks its advance towards the river, and by back pressure causes the water to rise vertically in the ground for a long distance back from the river, thereby reducing the inclination of the water-table and finally reversing the slope should the rise continue. During the existence of reversed slope of the water-table, water from the river forces itself into the gravel-beds and drives back the ground-water in a retreating wave. Thus retreating and advancing waves of ground-water, gradually tailing out as they advance inland, beat time to the fluctuation of the river. Certainly the influence of rivers upon storage of water in the ground is pronounced, and is a factor of support which can be depended upon in years of abnor-

mally low rainfall so long as the gravel deposits extend well below the river-bed and afford permanent storage.

Whatever support such deposits of gravel receive from the river naturally may be expected to operate when artificial conditions are created by drawing water from the ground storage reservoirs to whatever depth may be found practicable. The river inflow to restore depleted storage is pronounced and effective and proportionately eliminates the necessity of considering extensively the catchment area and the amount of rainfall. The velocity of flow of such rivers as the two referred to is sufficient to prevent a silting of the bed of the river, inasmuch as it is naturally much in excess of any percolating velocity which can originate in the passage of river-water downward through its bed to supply any near-by artificially depleted storage.

Sluggish and muddy streams may accumulate a layer of silt over the bed which at ordinary stages of water interferes with replenishment of depleted storage in neighboring deposits by inflow from the stream. In such instances the basis of computation of available yield must be based largely upon absorbed rainfall. Likewise the basis of computations for the sand and gravel deposits often encountered in territory bordering the coast must be the absorbed rainfall.

Thus it is found in these instances that a mechanical analysis of the water-bearing material, its depth and extent, the natural slope of the water-table and the velocity of underflow, are important factors for the determination of permanent yielding capacity. The slopes of the water-table and accordingly the velocity of underflow will be found to possess seasonal and periodical variations, often compelling a draft upon storage for a considerable period of time to furnish the desired yield. It is important to determine the probable extent to which natural storage may be thus drawn upon without permanently depleting it; that is to say, without exceeding the replenishing capacity of the rainfall during the wetter periods.

The dry-weather flow of streams is an index of the volume of underflow, and therefore a careful gauging of those streams which drain a limited territory affords information of the rainfall-absorbing characteristics and yielding capacity of restricted areas.

Such studies are best adapted on a limited scale to streams draining comparatively small catchment areas through which the meteorological conditions are generally the same. However when facilities admit of extensive observation a measurement of the increment by which the flow of a stream progressively increases, affords information of the volume of underflow entering even large streams draining extensive territory.

A hydrological survey of territories and districts with a view of determining underground run-off of rainfall is a question which has engaged the attention of but few communities in this country. Therefore when a specialist is called upon to give advice with regard to the feasibility of a ground-water supply, there is usually a dearth of information, except that of a very general character, often confined solely to rainfall statistics collected by the meteorological department of the general government. The gauging of streams and other necessary accompanying measurements or observations in order to secure complete data of the locality is a matter which usually cannot be undertaken in the limited time available when a project for water-works is first carried out, and as a rule the first steps are usually guided by and based upon an analysis of such general information as can be gathered, together with the facts and data bearing upon the subject which may have been collected elsewhere. The experience of communities with ground-water supplies is often very disappointing—perhaps no more so with ground-water than with other sources of water-supply—but nevertheless serious and tangible enough to affect the interest of the entire community and to arouse doubts of the expediency of making further efforts of improvement along the line heretofore pursued. A systematic water-works and water-supply record maintained from year to year showing in detail the technical history of the water-supply plant would serve greatly in solving many perplexing local problems. Unfortunately such records are not always available, and frequently after years of experience only meager general information is still all that is available to guide in an improvement of the water-supply works. However, where detailed information is lacking, it is always profitable to supply its place by such well-directed tests and observations as time and facilities may admit. The nature of

some of the detailed information that is desirable and which can be acquired by comparatively inexpensive tests has been already pointed out. But in addition to this information a sanitary survey of the site of a proposed ground-water supply should be made to determine both the area and physical make-up of the catchment basin, particularly when the balancing effect of a near-by large river upon the ground-water storage is not available. In this case rainfall must be the sole reliance of a ground-water supply. The soil in the catchment basin which collects the rainfall must be porous enough to absorb a considerable percentage of the water and the geological substructure of a character which admits of the necessary free lateral movement of the ground-water towards the site selected for developing the water-supply.

There is a wide difference in absorbing capacity of the soils and sands, a difference which is well exemplified in the experiments of Stearns, before alluded to on a previous page of this chapter. Upon tabulating the results of these experiments in a manner to show the rate of vertical percolation (which is about ten times the rate determined by the experiments at a grade of 1 in 10) some idea may be formed of the relative porosity of sand and soils.

PERCOLATING CAPACITY OF SOILS.

Material.	Rate of vertical absorption in feet per day.	Ratio of absorption.
Coarse sand.	293.3	4314
Medium sand	53.3	784
Fine sand	12.0	176
Very fine sand.	0.96	14
Soil.	0.068	1

The results of another experiment by Professor King to determine the rate of percolation through a sandy soil covered with a yellowish sand are given in the table on page 44.

We must remember, however, that the laboratory experiments to determine the rate of percolation through unassorted sand and soils do not take into consideration the modifying effects of natural conditions such as frost action, vegetable growth, the intermittent and variable intensity of the application of water

which occurs naturally through the rainfall; in both these experiments the water was fed continuously to the material experimented with. Relatively the experiments are valuable suggestions and seem to show that a collecting area which is desirable for the absorption of rainfall should be of a sandy texture. The close-grained soils where clay generally predominates are controlled largely by the laws of capillarity and admit of but an exceeding slow rate of percolation.

PERCOLATING CAPACITY OF SOILS.

Section of pit.	Effective size of soil grains in millimeters.		Percolation per 24 hours in inches of water.
	Lower soil.	Upper soil.	
1	0.0338	0.2844	6.7
2	0.0414	0.2844	10.104
3	0.0405	0.2844	12.151
4	0.0526	0.2844	12.678
		Average...	10.408

In field operations only a fraction of the rainfall is absorbed by the underflow even in porous soils, the balance disappearing as surface run-off or evaporation and absorption by plant life.

In irrigated sections in Colorado, Professor L. G. Carpenter's measurements show a return to the rivers of 30 per cent or more of the water applied for irrigation where the depth of the annual application of water to land aggregates about 30 or more inches; the measurements also indicate a return to the river by underflow of 1 cubic foot per second constant flow for each 1000 acres of irrigated land for the Poudre district; for the Upper Platte district 1 cubic foot for each 430 acres, and in the Lower Platte the same amount of return water for each 250 acres, or corresponding amounts expressed in another unit of measure of 414,720, 964,480, and 1,658,880 gallons per day per square mile, respectively. The difference, Professor Carpenter states, "is due mostly to the greater distance for the seepage to reach the main stream, and to the times and amount of water applied." It has taken many years for the seepage water from irrigated districts to fill the underground storage reservoirs and correspondingly to raise the water-table sufficiently to produce the above-

stated underflow into the rivers, and doubtless an equilibrium is not yet altogether established. In some instances the water-table has been raised forty feet or more.

It is impossible to tabulate the underground run-off per unit of area to serve as a guide in estimating ground-water yield except in particular cases and for local guidance only. In some instances where the collecting area is large and flat and the soil is permeable in a rather humid climate, about 30 per cent of a minimum annual rainfall and 50 per cent of the maximum annual rainfall, or about 40 to 45 per cent of the average annual rainfall, is considered to be a safe basis of estimate. The average rainfall is perhaps a safe basis of estimate where there is extensive underground storage for the reason that this storage can be drawn upon for deficiency of yield during the periods of abnormally low rainfall. Thus for an average rainfall of 38 inches covering a period of 50 years of record, 40 per cent, or 15 inches, may be considered available underground run-off, amounting to an average daily yield of about 716,000 gallons per day per square mile.

In order that this volume of water may pass away freely without water-logging the surface soil, there must be a sufficient depth of porous gravel formation beneath to admit of free lateral movement of the water and a free outlet. Thus upon the assumption that the body of moving water must pass through a layer of sand one mile long we find by computation, allowing one-third the volume of sand to be void space, that in order to discharge 716,000 gallons per day the sand layer should be at least 54 feet thick for a velocity through the sand of 1 foot per day, 27 feet for a velocity of 2 feet, about 18 feet for a velocity of 3 feet, etc., requiring respectively a water-table slope for a No. 50 sand, which is an average water-bearing sand, of 8, 16, and 24 inches, respectively, per 1000 feet. But in a very expansive flat valley where there is little opportunity for surface run-off, also where there is only a comparatively thin layer of water-bearing sand, say 18 to 20 feet in depth, the slope of the water-table must necessarily be comparatively light, in fact so light that a water-logged soil would result were the underlying sand no coarser than the No. 50, which has been assumed. The

heavier slopes above computed could only obtain in localities where there is corresponding ground-surface slopes, as in the sandy coast territory.

These examples serve to illustrate the relation of the various factors which it is necessary to consider in estimating ground-water yield with a reasonable degree of accuracy. But the matter may be carried a step further, and the relation of these several factors with the length of development required in order to collect the ground-water may be shown in the form of a diagram as in Plate V on opposite page.

The diagram contains lines of slope for various grades of sand, a line of velocity in terms of the voids in the sand assumed to be practically one-fourth of the volume of sand, and a curve of length of development required to furnish 1,000,000 gallons of water per day for stated rates of yield per lineal foot of development from a sand stratum 35 feet in depth measured from the water-table downward. The 35 feet of depth of saturated sand is purely an arbitrary depth and is considered simply because it is the actual minimum depth of a saturated gravel in the particular case for which the diagram was originally prepared. In the diagram the line of water-supply development is assumed to have a positive feed from both sides, hence the stated rate of yield is double the yield from one side only. The application of the diagram is readily understood. Having made a boring on the site of a proposed water-supply development and by several analyses determined that the average effective size of the water-bearing sand is No. 40, and by levels of the water surface of various convenient wells in the vicinity, either bored for the purpose or domestic wells in an undisturbed state, having found that the water-table has a slope of 1 foot in 1000 feet, we start at the 1-foot mark on the slope scale of the diagram, move horizontally across the diagram to an intersection with the slope line of a No. 40 sand, then trace vertically the line of intersection with the length of development curve, then horizontally from that point of intersection to the development scale and read the length there indicated; the same vertical line carried to the velocity line, then to velocity scale, will give the velocity of flow in terms of voids in the sand,

PLATE V.

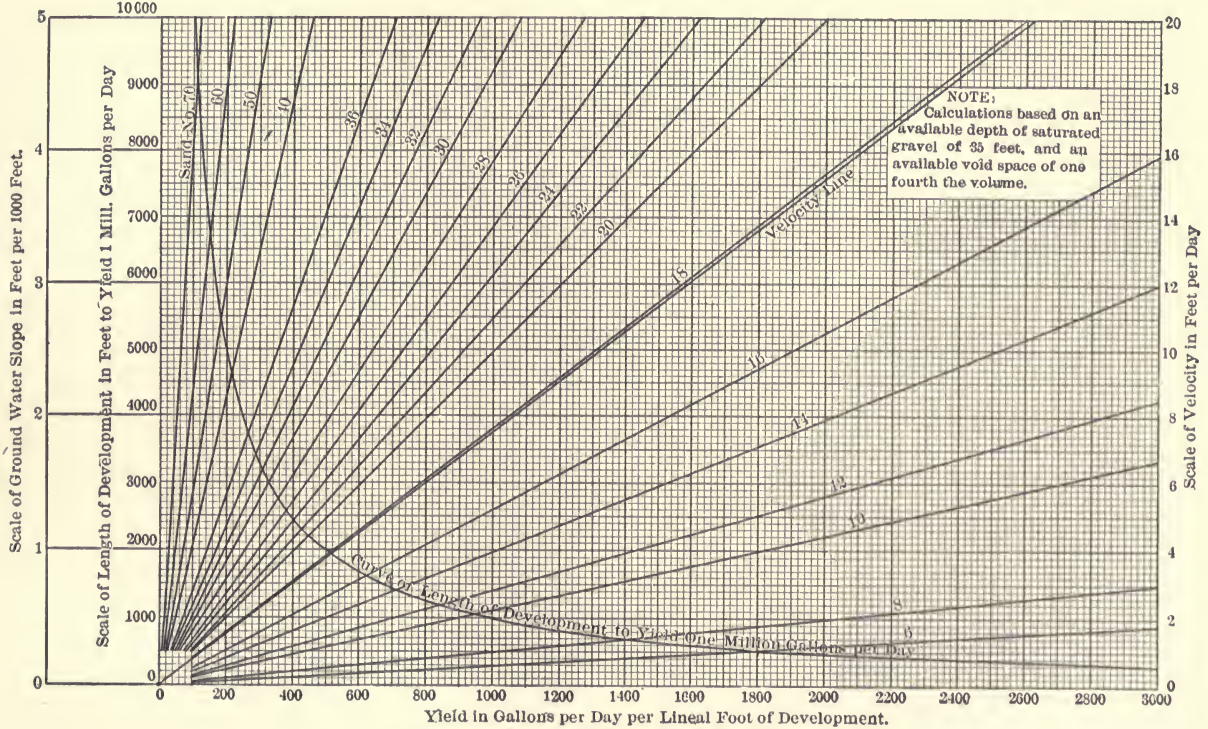


Diagram Relating to Ground-water Supply.

and also if carried to the bottom of the diagram will give the yielding capacity of the sand from two sides per lineal foot of development for the stated depth of 35 feet. Should the depth of water-bearing sand be either more or less than the assumed 35 feet of depth, the yield of water per lineal foot of development as ascertained from the diagram in the manner above described should be multiplied by the ratio of the actual depth to 35 and the point of the development curve vertically above the corrected yield per lineal foot will then give the desired length of development. In other words, the yielding capacity of the sand is thus approximated, but whether the actual feed is sufficient to support such a yield is a question which must be determined by a survey of the catchment area and examination of the porosity of the surface soil.

The use of the diagram and other data herein given is designed as a guide to the judgment in estimating the yielding capacity of a territory selected as a suitable site for a ground-water supply. Of course where supply works are already developed and in use for several years the yielding capacity of the sand-bed and its supplying territory is measured by the amount of water actually pumped. There is no adequate substitute for such a positive test, *provided the mechanical means of taking the water out of the sand is in nowise throttled*, and provided the physical data bearing upon the effect of the pumping upon the ground-water table are collected and carefully studied. In the absence of such information there is no way of testing the full yielding capacity of the supplying territory until the rate of pumping shall have progressively increased up to the breaking-point, or that of absolute refusal of the ground to deliver the water in the volume required by the pumps. This experience often puzzles a water-works management and may lead to a dilemma if technical studies have not been pursued in anticipation of a demand for extensions and improvements.

The development of ground-water is accomplished in various ways, but the simplest way is probably the single open well. Of this method there is little to be said because of its simplicity, except that it is the method often adopted by small towns and occasionally under exceptionally favorable circumstances by

cities of considerable size. The open well sometimes serves the triple purpose of storage-well, receiving-well for galleries or a gang of tubular wells, as well as for individual water-supply purposes. An example of this kind is illustrated in Plate VI, which shows a small open well receiving the siphon pipes from two lines of tubular wells. The development lay at the foot of a hill bordering a piece of swampy ground supplied by springs and was projected with a view of intercepting the underflow feeding the springs referred to. The supply of water afforded by the wells was small, about 100,000 gallons per day, but sufficient at the time for the supply of the village for which it was developed.

Wells of this kind, when penetrating deeply a heavy and extensive bed of water-bearing gravel, supply water in large quantities, but may become rather expensive disappointments when they simply uncover a thin layer of sand or penetrate what may be termed a pocket of gravel. In the one instance the depth of supplying sand is too small to admit of either a sufficient storage or a flow of water; in the other instance the pocket of gravel yields water bountifully at first, but finally fails as the stored rainfall representing perhaps years of accumulation becomes depleted. A case to the point is an open well 30 feet in diameter and 40 feet deep which uncovered a thin layer of sand about 3 feet deep under a deep heavy soil. The capacity of the well was but 126,000 gallons per day even when reinforced by two 6-inch holes drilled into sand rock underlying the bed of sand. In several other instances wells 30 to 50 feet in diameter completely failed within a year because of either an insufficient supplying area or an exhaustion of a sand-pocket, notwithstanding the fact that a short-time pumping test developed the presence of considerable water. Failures of this kind are frequent with small water-works, but with some degree of justification when insufficient funds do not admit of seeking and developing a more distant but more permanent source of water-supply.

Similar in character and almost as simple as the open well is the filter-gallery which parallels a stream for the purpose of intercepting and collecting the underflow through a body of gravel. Like the open well the exposed gravel bottom of the

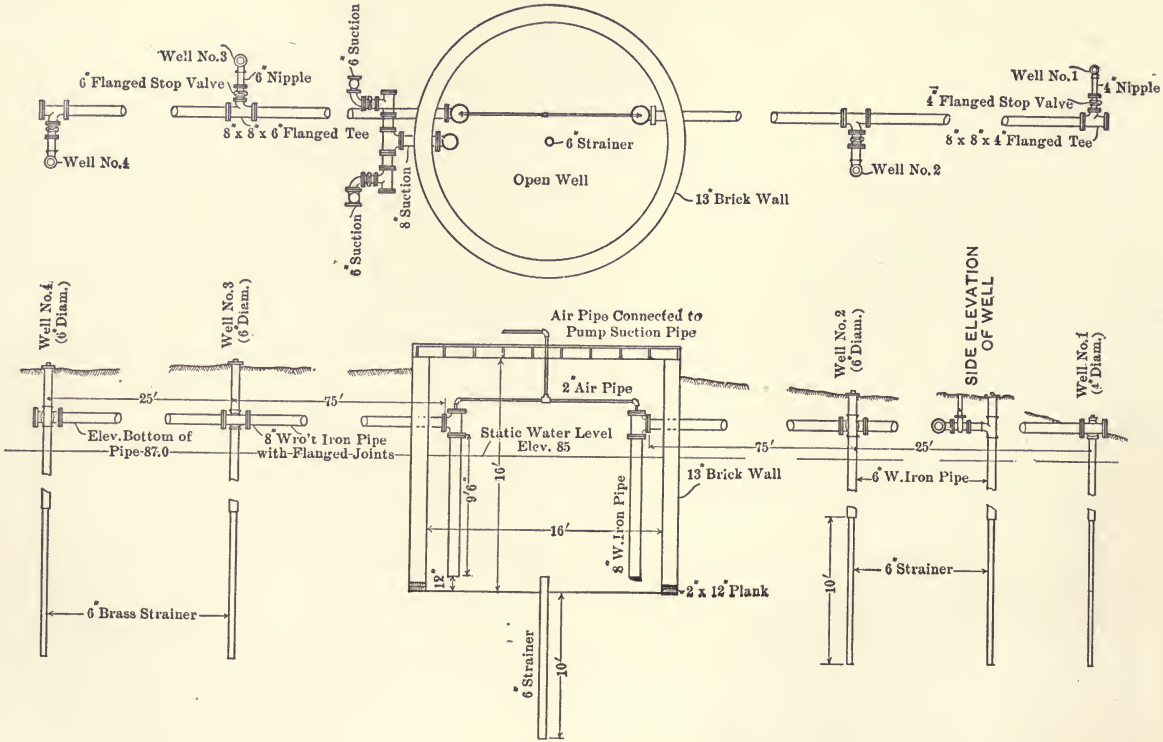

Well No.3
6" Flanged Stop Valve
6" Nipple
8" x 8" x 6" Flanged Tee
Well No.4
6" Suction
8" Suction
6" Suction
6" Strainer
Open Well
13" Brick Wall
Well No.1
4" Nipple
4" Flanged Stop Valve
8" x 8" x 4" Flanged Tee
Well No.2
Well No.4. (6" Diam.)
Well No.3 (6" Diam.)
Air Pipe Connected to Pump Suction Pipe
2" Air Pipe
25'
75'
Elev. Bottom of Pipe-87.0
8" Wro't Iron Pipe with Flanged Joints
Static Water Level Elev. 85
16'
9'6"
19"
8" W. Iron Pipe
13" Brick Wall
2" x 12" Plank
6" Brass Strainer
6" Strainer
10'
Well No.2 (6" Diam.)
SIDE ELEVATION OF WELL
Well No.1 (4" Diam.)
6" W. Iron Pipe
6" Strainer
10'

PLATE VI.

gallery furnishes the supply of water which is delivered into a receiving-well within convenient reach of pumps or into a gravity pipe connecting with some distant reservoir. Galleries of the latter kind connected with a reservoir of large capacity may run continuously, but should the rate of discharge exceed the rate of underflow replenishment a gradual depletion of ground storage must ensue and eventually a gradual reduction of yield must follow. The reason of this is plain when it is considered that the maximum head or pressure of water actuating the delivery into the gallery depends upon the depth of the gallery below the water-table and the delivery depends upon the porosity of the gravel. Whenever a continuous discharge from the gallery causes the water-table to fall progressively, replenishment is evidently exceeded, the actuating pressure falls and proportionately the rate of yield of the gallery decreases until an equilibrium between replenishment and yield is established.

A comparatively thin bed of coarse sand and gravel not far from the surface of the ground is the most favorable condition for gallery development. The gallery should be located as far as practicable below the water-table in order to utilize to the fullest extent the storage which a gravel deposit affords for occasional heavy draft of water.

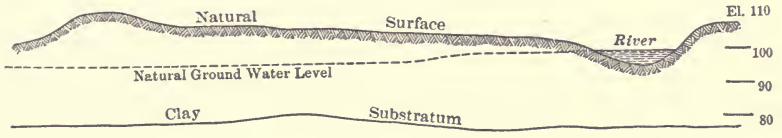


FIG. 2.

Fig. 2 illustrates the depth of a deposit of water-bearing gravel which has furnished the water-supply of a large-sized city for over fifteen years.

The diagram shows the normal ground-water table to be about 18 to 20 feet above an impervious clay substratum. The water-supply is collected in a system of galleries aggregating 2340 feet in length leading into a combined water-supply and suction well about 47 feet in diameter containing in the aggregate a gross infiltration area of 12,500 square feet. At the time of which we are speaking, the average daily consumption was nearly 3,000,000

gallons per day and infiltration was at the rate of 240 gallons per square foot of gross area or about 300 gallons per square foot of net area. This rate of infiltration has been increased fully 60 per cent upon occasions requiring heavy fire service.

A water-level test with the gallery delivering nearly 3,500,000 gallons of water showed a drop at the gallery of $3\frac{1}{2}$ feet below normal water-table level and an actuating slope of the ground-water of about 7 feet per 1000 feet, corresponding to a lateral velocity of approach of about 6 feet of solid water per day through the $16\frac{1}{2}$ feet of saturated gravel near the gallery, assuming water to be supplied equally from both sides of the gallery. The average coefficient of porosity is estimated at 1000 and the effective grade of the sand a No. 36. We regret to have no mechanical analysis of the sand, but if the curve of the water-table under draft be compared with the Muscatine curve as shown by Fig. 3, the higher resistance of the sand referred to becomes apparent.

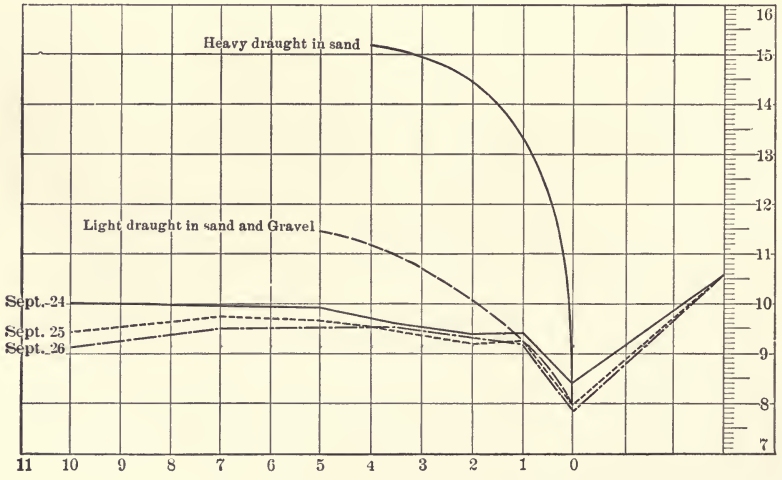


FIG. 3.—Diagram of Ground-water Curves.

The level of the bottom of the galleries ranges from 8 to 9 feet below the normal water-table, leaving several feet of ground storage between the gallery and the clay substratum which cannot be drawn upon in emergencies. The importance of making this deeper storage available justifies the construction of a gallery somewhat deeper than the original galleries whenever the demand

for water-supply extension arises. The expense entailed in carrying out this work may be comparatively heavy, but nevertheless not so great as that of providing equivalent emergency surface storage, which would preserve unimpaired the quality of the ground-water. Possibly the local conditions warrant this form of water-supply development above others; at any rate the success of the original galleries in supplying so large a volume of water as 1,000,000,000 gallons or more annually must certainly inspire sufficient local confidence in the form of construction as to justify the preference.

It will be noted in the preceding illustration that we have made an estimate simply of the lateral velocity of the flow of water through the sand as it approaches the gallery to supply the draft of water therefrom, and not of the vertical velocity of infiltration through the gravel at the bottom of the gallery, for the reason that it is the permissible velocity of approach to the gallery that governs the rate of infiltration. In turn the velocity of approach is controlled by the grade or porosity of the sand through which the water must flow in approaching the gallery.

So far as we know there is no record of measurements taken to ascertain the influence of the river adjoining the gallery upon the yield of ground-water, nor yet of any examination or survey of the extent and general physical characteristics of the water-bearing gravel underlying the river valley. Thus far the large amount of water which has been drawn from this gravel-bed does not appear to have affected the general characteristics of the water-table, which fluctuates with the rise and fall of the adjoining stream. It is a case where no interest dependent upon the source of water-supply seems to have suffered thus far from a neglect of making a thorough examination of the water-supplying capabilities of this particular locality. However, the importance of the city and the magnitude of the investment now at stake in this case, as well as in any other similar case, should suggest to a prudent water-works management the desirability at least of making such examinations as admit of approximating the yielding capacity of the existing source of water-supply and thereby become informed of the probable limitation to be placed upon future development work.

Theoretically the limitation of filtering-gallery extension is

indefinite so long as due regard can be given to the hydraulic conditions which are necessary to produce the required flow of water within the gallery and to structural requirements. The rate of infiltration should not be so great as to disturb seriously the sand underlying the usual ground floor of the gallery.

The depth of gallery construction below the normal water-table possesses a very decided limitation from the standpoint of cost; in fact such construction is confined in this particular direction within such narrow limits that it is seldom considered a feasible method of water development in localities where deep water-bearing strata prevail, particularly when the coarse and more desirable stratum is near the bottom and much below the water-table. Even where the gravel-bed is shallow, as in the preceding illustration, the superiority of a gallery over other forms of development is often questionable. Upon this debatable point it is thought that much depends upon the mechanical make-up of the water-bearing material and upon the cost of maintenance of gallery construction compared with other forms of construction.

For example consider what other form of water-supply development can be substituted for an infiltration gallery, in the deposit of water-bearing material of which Fig. 2, on page 51, is a sketch. Several large open wells connected in series by a gravity-flow pipe and finally with a pump-suction well could scarcely prove an adequate substitute because the expense of constructing the wells and the connecting gravity pipe lines would fully equal the cost of a gallery of equal yielding capacity. This fact seems clear upon the face for the reason that all the conditions affecting the yield and the cost of construction and maintenance are identical in the two cases. But were the series of wells connected by a series of siphons of gradually increasing capacity as the suction well is approached some saving in the investment would result, although the cost of operation and maintenance of the water-supply works would be somewhat increased because of the necessity of having to provide and maintain a means of removing air from the series of siphons. The uncertainty of a prompt response of a series of siphons to the fluctuating requirements of consumption and the inaccessibility of the several siphons would militate against this method of development even at a reduced cost of construction.

A more unique form of construction, shown by Fig. 4, would be to construct a main siphon pipe parallel with but to one side of the series of open wells and on a continuous rising grade to the suction well where it should terminate in a vertical down-take pipe dipping several feet below low water in the suction well and capped

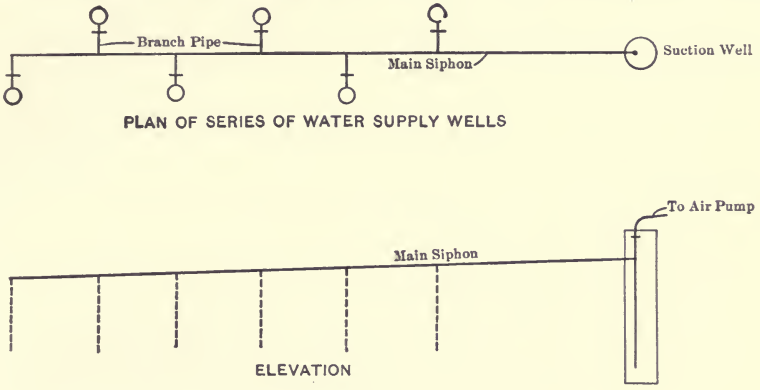


FIG. 4.

at the top by a large vacuum-chamber from which entrailed air could be exhausted by means of a vacuum-pump. Each open well should then be connected with the large siphon pipe by means of a branch pipe of sufficient capacity to accommodate the yield of the well in the manner shown by Fig. 5.

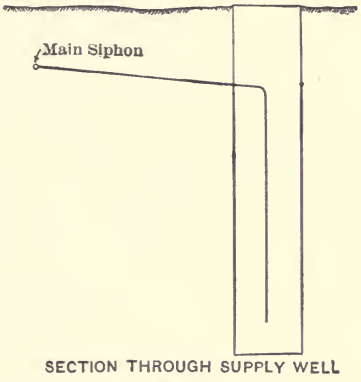


FIG. 5.

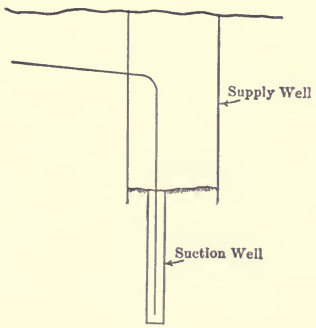


FIG. 6.

The objections to multiple siphons in the former arrangement of open wells are altogether overcome by the arrangement just

described. There is only one siphon and only one point in this one siphon where there is any possible chance of an air-lock, namely, at the vacuum-chamber on or near the down-take inlet to the suction well, and then only through neglect of the attendant at the pumping-station. The vacuum in the siphon pipe can always be maintained and the water-supply works can always be in shape to respond to any demand of consumption up to the supplying capacity of the wells. The only chance for a break is through the exhaustion of the supply of water in any one well to the extent of exposing the mouth of the branch supply pipe to the siphon, but this danger of a break of the vacuum may be avoided by a small suction well sunk inside the supply well to some depth below the limit of suction as indicated by Fig. 6.

This form of construction usually requires less investment in water-supply works than does an infiltration gallery. The water-supply wells may be sunk with proper appliances by dredging the material from the inside as the curb descends through its own or a superimposed weight, thereby permitting all construction work to be accomplished above the ground-water table with but little if any pumping for construction purposes, which is the chief source of expense in the construction of an infiltration gallery. The cost of operating and maintaining the well system of supply should be no greater than that of the gallery system of supply. Besides a series of wells can be more readily sunk to a depth which will make available practically all of the permanent storage in the water-bearing material, as would be the case in the illustration referred to were the shoe of each of the supply wells to penetrate the water-bearing material to a level 4 or 5 feet above the clay substratum.

The general arrangement would perhaps appear more balanced were the supply wells distributed on either side of the siphon pipe, even though in the matter of yield there might be no material difference, the wells being naturally distributed in a direction normal to the slope of the natural ground-water table.

Consideration of the matter of an economical substitute for an infiltration gallery may be carried a step farther by attempting to find a satisfactory substitute for the large open well. This step naturally leads to a consideration of a series of small tubular

wells ranged along either side of the large siphon pipe which has just been described. These small wells may be either open or closed, driven or bored, according to present practice. Imagine two rows of small wells, one on either side of the siphon pipe above described and close enough together to absorb the entire flow of water that the water-bearing material is capable of passing at the maximum permissible slope. For the purpose of ascertaining what the slope should be to produce a maximum discharge, or, in other words, for the purpose of defining the limit to which the water-table should be depressed near such a system of wells as has been described in order to produce a maximum velocity of approach through the sand and accordingly a maximum discharge, reference is made to Fig. 7 and the following solution.

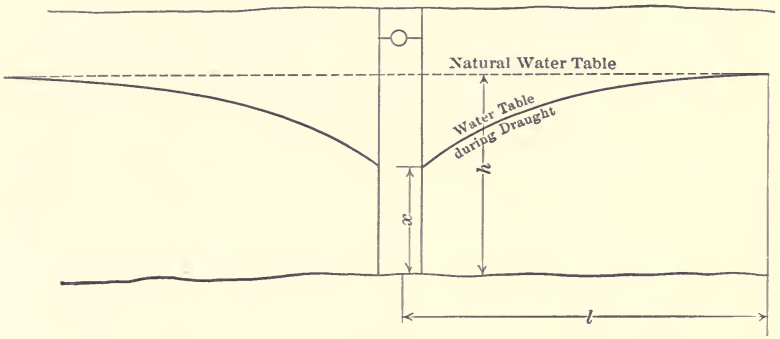


FIG. 7.

Let v = velocity of approach of the ground-water = ki;

i = the slope of ground-water = $\frac{h-x}{l}$;

h = the depth from the natural water-table to substratum;

x = " " " " draft " " "

l = the distance from wells to the point where the draft water-table meets the natural water-table;

a = area of saturated section at or near the wells = x times unity for a unit length of section;

Q = the discharge for 24 hours = av;

hence $Q = av = aki = xki$ for unit length of section.

Substitute in value for Q the value of $i=\frac{h-x}{l}$,

then $$Q=\frac{k}{l}x(h-x).$$

Now the condition which makes the factor $x(h-x)$ a maximum is the condition which gives a maximum value for Q and accordingly a maximum velocity of approach through the sand. It is found that the value of x which makes this factor a maximum is $\frac{1}{2}h$.

Hence, by substitution,

$$Q=\frac{k}{l}(h-\tfrac{1}{2}h)\tfrac{1}{2}h=\frac{1}{4}\frac{k}{l}h^2 \text{ in cubic feet per day.}$$

In order to reduce to a discharge in gallons per 24 hours, considering the water to approach equally from both sides of a system of wells, multiply the above equation by 2 and 7.5, making

$$Q=3.75\frac{k}{l}h^2.$$

Reference has been made to an illustration, Fig. 2, where $h=20$ feet and $k=1000$, and where the observed value of l was about 500 feet; hence for that case

$$Q=\frac{3.75\times 1000\times 20\times 20}{500}=3000 \text{ gallons per day.}$$

That is to say this computed rate of discharge is the maximum discharging capacity of the stated grade of sand under stated conditions of slope and depth of water-bearing material independent of any question of rainfall or river support, and is representative of the conditions which make the best use of the available ground storage for emergency draft.

It is evident that a gallery which is sufficiently low in the ground to depress the natural water-table over it one-half the depth of the water-bearing stratum receives the maximum rate of infiltration. But when a system of wells is substituted for a gallery, the direct current of approach must break up into a

series of converging currents of rapidly increasing velocity as each well of the series is approached. The larger the number of the wells in a given distance the less becomes the several individual areas of converging flow, until finally with the small wells practically in contact the gallery conditions become duplicated in effect. When the wells are a considerable distance apart, as they usually are, the quantity of water entering the strainer of any individual well must equal the quantity which approaches it by direct flow and feeds the zone of the converging current which feeds the well, consequently the quantity of water flowing in a prism of a length equal to the distance between the wells and a depth equal to one-half the normally saturated area divided by the exterior area of the strainer gives the velocity of flow from the sand in contact with the strainer subject to such correction as the relatively small area of the strainer openings and well interference may require.

Strainers. — The ordinary well-strainer possesses but about 13 to 15 per cent of the entire metal surface as openings, and not more than one-third of this per cent is available for use, consequently the velocity of flow through the openings must be very much more than the velocity of flow through the sand at the surface of contact of the sand with the strainer.

There are certainly some well-defined principles upon which the area of the openings in, and the length of, a strainer should be based.

Evidently the openings should have a form which offers the least resistance to the flow of water and a size which can absorb the water as fast as it is delivered from the surrounding sand. Of course the geometrical form of opening which offers the least resistance to the flow of water is the circle. But while practical considerations sometimes require a departure from the circular form of opening they do not require a proportionate departure from the effective discharging capacity of that form of opening; for instance, a departure from a circular to a narrow rectangular opening should be accompanied with a compensating increase of area of the slot in proportion to its greater resistance to the flow of water.

Usually the head or pressure which actuates the flow of water

through the strainer opening into the well is the negative head produced by pumping. Whenever water does not enter the well fast enough through strainer openings of insufficient area the pump gradually increases the vacuum in the well, within practical limits of course, in an effort to secure a feed of water. The resulting but gradually increasing pressure outside the strainer eventually packs the sand about the strainer to the extent of throttling the already deficient area of openings and proportionately the feeding capacity of the strainer. A system of wells thus throttled will usually be found to yield a gradually decreasing volume of water and finally to decrease the feed to the pumps to such an extent as to render the method of water-supply development more or less of a failure.

This throttled condition of strainer is often the direct result of the use of a strainer with openings so small that the fine sand cannot enter the well. The theory forming the basis of this practice may be correct when the sand in which the strainer is embedded is generally fine and of uniform texture, but it cannot hold good where the layer of sand is an assorted mixture in which fine sand is a comparatively small percentage of the bulk, for in that case the strainer should be coarse enough to allow the finer sand to pass through the openings into the well and to be removed mechanically. In fact it is often a most excellent practice to give each well a preliminary washing by means of a directly attached pump operated at a rate of pumping somewhat in excess of that which would prevail in general service. Any portion of the sand thus drawn through the openings of the strainer and not ejected from the pump with the water can readily be withdrawn with a sand-bucket. The remnant of coarse sand about the strainer prevents the subsequent advance of fine sand in the same manner that the graduated gravel layer of a sand-filter prevents sand entering the underdrain.

The danger of further movement of fine sand after such a washing is very remote because of the rapidly decreasing velocity of flow through the sand as the distance from the center of the well increases. In order to determine what this velocity may be and the law by which it increases as the well is approached it is necessary to devise a mathematical expression embracing

the law or principle of radial flow. Assume that the water flows in a radial direction towards the well through a series of concentric cylinders $C, C_1, C_2 \ldots W$ (Fig. 8) of a depth $x, x_1, x_2 \ldots x_w$, measured from the impervious stratum YY to the water-table assumed by the approaching water. Throughout each concentric cylinder the velocity and velocity head are constant and are inversely proportional to the distance from the center of the well. Moreover, the velocity through any cylinder C multiplied

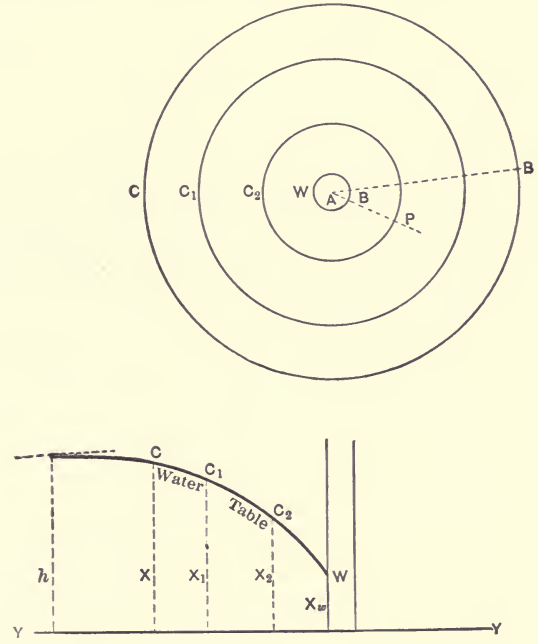


FIG. 8.

by its distance from the center of the well is equal to the velocity of flow through any other section C multiplied by its distance from the center. Hence by representing the law of variation of velocity by the general formula $\rho' dv = v' d\rho = v\rho = \text{constant } (c)$, where ρ equals any distance from the center, the following general formula may be written:

$$v = \frac{c}{\rho}. \quad \ldots \ldots \ldots \quad (1)$$

It is also known that the quantity of water which flows through any cylinder C must also flow through any other cylinder C_1, C_2.

In the general expression for velocity $v=ki$, where $k=$the factor of resistance and i is the slope which for any infinitely small arc of the curved water-table of radial flow may be expressed by $\frac{dx}{d\rho}$. Hence

$$v=k\frac{dx}{d\rho}.* \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

By substituting the value of v of equation (1) in equation (2) there follows:

$$\frac{c}{\rho}=k\frac{dx}{d\rho} \quad \text{or} \quad c\frac{d\rho}{\rho}=kdx,$$

or

$$c\int\frac{d\rho}{\rho}=k\int dx=c\log_e\rho=kx+C. \quad . \quad . \quad . \quad . \quad (3)$$

Now if the distance from the well to the boundary where the water-table of approach to the well meets or becomes the natural water-table be represented by R with its corresponding value of $x=h$, the natural depth of the water-bearing material, and the distance from the center to the exterior of the well by r, where the value of $x=x$, we may substitute these values in equation (3) and solve for C. Hence

$$\begin{aligned} c\log_e(R+r)-kh&=C,\\ c\log_e r-kx&=C. \end{aligned}$$

Now placing the values of C equal to each other there follows:

$$c[\log_e(R+r)-\log_e r]=k(h-x).$$

$$c=\frac{k(h-x)}{\log_e\left(\frac{R+r}{r}\right)}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

* The theoretical portion of this demonstration is suggested by Prof. Slichter's analysis of radial flow in Nineteenth Annual Report of U. S. Geological Survey.

Substituting the value of c of equation (4) in equation (1) an expression for velocity is found:

$$v = \frac{k(h-x)}{\rho \log_e \left(\frac{R+r}{r}\right)}. \quad \ldots\ldots\ldots \quad (5)$$

For the particular case of velocity or discharge at the circumference of the well let $\rho = r$ and $Q = av = 2\pi rxv$, and by substitution in equation (5) an expression for discharge follows:

$$Q = \frac{2\pi kx(h-x)}{\log_e \left(\frac{R+r}{r}\right)}. \quad \ldots\ldots\ldots \quad (6)$$

Since the expression for Q is in terms of the variable x in the term $x(h-x)$, there is some value of x which makes this term and consequently Q a maximum.

Let $u = x(h-x)$,

then $du = hdx - 2xdx$.

$$\frac{du}{dx} = h - 2x = 0 \quad \text{or} \quad x = \tfrac{1}{2}h.$$

$$\frac{d^2u}{dx^2} = -2.$$

Hence the above term and consequently equation (6) become a maximum when $x = \frac{h}{2}$.

Now by making this substitution in equation (6) and reducing, there results

$$Q_{\text{max.}} = \frac{1.57kh^2}{\log_e \left(\frac{R+r}{r}\right)}. \quad \ldots\ldots\ldots \quad (7)$$

A similar substitution of $x = \frac{1}{2}h$ and $\rho = r$ in equation (5) gives a value of velocity corresponding to a maximum rate of discharge at the well circumference as follows, wherein d = diameter of well:

$$V_{\text{max.}} = \frac{kh}{d \log_e \left(\frac{R+r}{r}\right)}. \quad \ldots\ldots\ldots \quad (8)$$

In formulæ (4) to (8), inclusive, the natural logarithm ($\log_e$) is used and may be found by multiplying the common logarithm by 2.3. The reduction is made in the formulæ which follow and the logarithm indicated by c.l.

Formulæ (5) to (8), inclusive, should be multiplied by a factor of resistance of flow from the sand through the strainer, and so far as investigations have gone they point to a range of values of 0.17 to 0.25. Accordingly the reduced formulæ become

$$Q_{\max.} = 0.114 \text{ to } 0.17 \frac{kh^2}{\text{c.l.} \left(\frac{R+r}{r}\right)} \quad . \quad . \quad . \quad . \quad . \quad (9)$$

$$V_{\max.} = 0.073 \text{ to } 0.11 \frac{kh}{d, \text{c.l.} \left(\frac{R+r}{r}\right)} \quad . \quad . \quad . \quad . \quad (10)$$

For general application formulæ (5) and (6) similarly corrected become

$$Q = 0.45 \text{ to } 0.68k \frac{x(h-x)}{\text{c.l.} \left(\frac{R+r}{r}\right)} \quad . \quad . \quad . \quad . \quad . \quad (11)$$

$$v = 0.146 \text{ to } 0.22 \frac{k(h-x)}{d, \text{ c.l.} \left(\frac{R+r}{r}\right)} \quad . \quad . \quad . \quad . \quad (12)$$

The above formulæ, (9) to (12), inclusive, when applied to the flow into small wells like the ordinary drive or tubular well may be still further abbreviated without appreciable error by writing for com. log. $\left(\frac{R+r}{r}\right)$, com. log. $\left(\frac{D}{d}\right)$, where D = diameter of the outer extremity of the zone of depression and d = the diameter of the well.

A still further reduction may be made of the formulæ for special application to tubular wells by assuming the average diameter of a tubular well to be $\frac{1}{2}$ of a foot and that the average radius of the zone of depression is 1000 feet, then the com. log.

of $2000 \div \frac{1}{2}$ becomes equal to 3.6. Substituting this value in formulæ (11) and (12) there results—

For tubular wells:

$$Q = 0.125 \text{ to } 0.19\, kx(h-x). \quad . \quad . \quad . \quad . \quad . \quad (13)$$

$$v = 0.04 \text{ to } 0.061 \frac{k}{d}(h-x). \quad . \quad . \quad . \quad . \quad . \quad . \quad (14)$$

For maximum velocity let $x = \frac{1}{2}h$.

$$Q_{max.} = 0.031 \text{ to } 0.0475\, kh^2. \quad . \quad . \quad . \quad . \quad (15)$$

$$V_{max.} = 0.02 \text{ to } 0.03 \frac{kh}{d}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (16)$$

Precise results need not be expected from these or any other formulæ relating to ground-water flow, but they serve to show the relation of factors and conditions and to aid the judgment in the study of ground-water conditions and in the design of water-supply works. Simple and approximate formulæ serve practical purposes much better than complex formulæ, even though the latter may be more accurate theoretically.

It should also be remembered that when the values k of the table on page 31 or 37 are used, together with the values of other unknown quantities expressed in feet, the result of the solutions of the formulæ are in feet or cubic feet per day of a solid column of water. Considerable departure from the results of these formulæ may be expected where the material which supplies the water to a well is fissured, thereby admitting of the approach of water through channels of much less resistance than the natural pores of sand and gravel. Channels of this kind are to be expected in a rock formation and are sometimes encountered in a gravel formation.

It does not necessarily follow from the discussions that the length of a strainer should be one-half the depth of the water-bearing material in order to take advantage of the maximum delivery of the sand. In some instances, as when the water-bearing material is of a uniform grade and not deep, and when the greatest possible yield of water is desired in emergencies, the stated length of strainer may be desirable. In other cases the strainer

may be considerably shorter, as, for instance, when it is desirable to confine the strainer to a coarse material underlying a comparatively fine material and to absorb the water in the superimposed fine material through the coarse material. Thus while the length of the strainer need never be more than half of the depth of the water-bearing material, it may be considerably shorter, depending upon the relative porosity of the several strata and upon the support which the yield of the supplying gravel receives from the rainfall or other sources.

In every instance, however, the diameter of the strainer should be one which will admit of perforations of a sufficient aggregate capacity, within the limits of the desired length, to absorb the water as fast as delivered from the sand at its maximum rate of flow. This statement is also true of a strainer of greatest length in a water-bearing material of practically uniform texture.

An approximate formula for the area of strainer openings may be derived in the following manner: Let the discharge through a rectangular orifice be represented by the formula

$$Q' = (cbd\sqrt{2gh'})\ F, \text{ in cu. ft. per sec.,}$$

where c = a coefficient of resistance of discharge = 0.6;

bd = aggregate area of strainer openings;

$\sqrt{2g}$ = effect of gravity = 8;

h' = velocity head = $h - x$;

F = a factor of reduction to discharge per 24 hours combined with a factor to cover blank space between slots and the reduction of area of slots resulting from contact of sand and gravel = $(F'F'') = \frac{1}{15} \times 86400$.

By substitution and reduction,

$$Q' = 27648bd\sqrt{h-x}.$$

Since the radial flow at the well must be equal to the flow through the strainers, we may place Q of equation (13) equal to Q' and solve for bd; hence

$$27648bd\sqrt{h-x} = 0.125 \text{ to } 0.19\ kx(h-x),$$

or $$bd = 0.00065 \text{ to } 0.001\ kx\sqrt{h-x}, \text{ in square inches,} \quad . \quad . \quad (17)$$

or $$(bd)_{\max.} = 0.00016 \text{ to } 0.00025\ kh\sqrt{2h}, \text{ in square inches.} \quad . \quad (18)$$

It should be observed that fine strainer openings possess a very low value of area and hydraulic mean depth and consequently a very low discharging capacity; for instance, the ordinary horizontally slotted strainer having slots 1¼ inches long by $\frac{1}{32}$ of an inch wide has an area of about $\frac{1}{26}$ of a square inch and a hydraulic mean depth of about 0.015 and a total area of nearly 3¼ square inches per lineal foot of each row of slots; the hydraulic mean depth of a vertical slot 12″ high and ¼ of an inch wide is about 0.125 or nearly eight times that of the small slot and the area 3 square inches. Since the discharge varies as the square root of the hydraulic mean depth, the relative hydraulic efficiency of the two slots is at least one to two and one-half in favor of the coarse slot.

Rock wells, or those which are formed by drilling into a water-bearing rock, are subject to the application of the principles of flow herein discussed when the rock formation is porous rather than fissured. As a rule the drill-hole in the rock is not cased any deeper than is necessary to get a proper location for a pump, if conditions require the use of a deep-well pump. The strainer is the exposed portion of the drill-hole extending below the casing and possessing a yielding capacity dependent upon its porosity and within limits upon the amount the water-table is depressed by pumping and upon the depth of penetration of the drill-hole into the water-bearing rock.

The yield of a rock well varies so slightly with an increase of diameter, as indicated by the formulæ on the preceding pages, that the diameter of the well is an insignificant factor in the question of yielding capacity. To what extent the seams and fissures facilitate the flow of water through a rock formation is purely problematical, but so far as observation goes they are numerous even in sandstone strata and constitute the principal avenues of flow. In one instance which comes to mind, a water-supply well 8 feet in diameter excavated through a dry shale uncovered at a depth of 125 feet a stratum of water-bearing sandstone in which the pressure was sufficient to carry the water vertically 10 feet above the top of the stratum. The well penetrated the sand rock 25 feet and was then bowled out to a diameter of about 20 feet. The water entered the well through seams and

crevices and freely through a 12-inch bore-hole penetrating 12 feet below the bottom. The yield of the well was never accurately measured, but was approximately 60,000 to 70,000 gallons per day, and after a short period of time seemed to come chiefly through seams, fissures, and drill-holes in or near the bottom, the sides of the well becoming quite thoroughly drained. Similar conditions of flow were observed in other wells and in a tunnel cut through sand rock. The physical conditions surrounding the flow of subterranean water through rock are found to be more complex than those governing the flow of water through sand and gravel deposits, and accordingly wider departures of the actual yield of water from wells, theoretically computed from formulæ, may be expected. However, there appears no good reason for the modification of the formulæ hereinbefore offered, at least in the general form, although the constants and relation of factors may be altered to suit localities where investigation has developed the controlling physical conditions.

This line of discussion might be pursued further were it not for the fact that rock wells are not the usual form of ground-water supply development except in a comparatively few favored localities where heavy artesian flow is available. Moreover, the expense of developing large quantities of water from rock formation supplying only non-flowing wells is heavy per unit of volume of water thus developed—in fact very much heavier than is the cost of developing an equal flow of water from sand and gravel deposits, and often greater than the cost of the purification works required to purify an equal amount of surface-water.

Returning to the subject of well-strainer, it may be stated that the form of strainer is an essential consideration in the development of ground-water supply and can in nowise be restricted to any particular standard of manufacture. The ordinary drive-well strainer, composed of a metal skeleton surrounded with wire gauze and a perforated sheath of brass plate, is sometimes injured seriously in driving and is altogether unsuited to a coarse water-bearing bed of sand and gravel. This form of strainer is usually employed for wells of 2 to 3 inches in diameter, and in this regard is open to the serious objection of being forced out of a vertical direction by an obstacle like a stone or buried log of wood;

in fact any form of drive-point strainer leaves one entirely in the dark as to the precise character of the material encountered in driving and accordingly without any knowledge whatever of the most suitable stratum in which to imbed the strainer. While the use of it serves a good purpose for the development of a small amount of water for individual uses, still there is scarcely any warrant for its application to the development of water for the public water-supply. What is needed in this regard is a form of strainer designed to suit the grade of material found in the most desirable substratum located by borings which expose the location and general make-up of all the substrata. Such a strainer may frequently be reduced to the simple form of a series of vertical slots $\frac{1}{8}$ to $\frac{3}{8}$ of an inch wide, cut through an ordinary wrought-iron or steel pipe jointed with the well-casing and an inseparable part thereof. The wide-slot strainer can frequently be used in material so fine that much of it can pass through the slots by simply depositing a layer of gravel between the strainer and the sand surrounding it during the process of construction. This may be accomplished by first sinking a casing to the impervious substratum several inches larger in diameter than the strainer, then, after lowering the permanent well-tubing with its attached strainer to the desired position inside the large tube, by filling beneath and around the strainer with clean gravel too large to pass the slots (see Fig. 9). Upon pulling the outer casing the layer of gravel remains between the strainer and the water-bearing sand and has the same relation thereto as the surface layer of gravel of a filter has to the sand above and the underdrain beneath it. The gravel can be deposited about the strainer progressively as the outer casing is withdrawn in order to offer less resistance and thereby facilitate the work of withdrawing the outer casing, which at the best is heavy work. The construction shown by Fig. 9 is adapted to deep water-bearing strata, but a modified form is practicable in thin coarse water-bearing material where for practical reasons it is necesary to use a short strainer of comparatively large diameter. Fig. 10 illustrates this form of strainer and represents a case where the drop-pipe is attached at one end to the strainer and at the other end to a siphon or suction-pipe under conditions where a serious air leak would be fatal to successful operation. The strainer must

be so short that the depressed water-table shall not come below the strainer openings, for the air which enters the exposed slots will destroy the vacuum formed by siphon action or by the pumps. The strainer attached to the drop-pipe through a reducing special has an open bottom resting on an artificially made bed of heavy gravel and absorbs water through large vertical slots. It may be made of ordinary wrought-iron or steel pipe.

Like the previous case the wells must be excavated through a casing-tube larger in diameter than the strainer, the bottom of

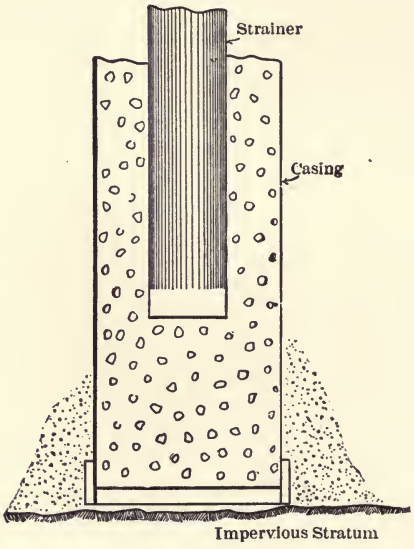


FIG. 9.

gravel deposited inside the tube, the strainer and drop-pipe adjusted to position, then the outer casing must be carefully withdrawn. The bed of gravel should be deep enough to allow of a free flow of water through the open bottom of the strainer as well as through the slots.

When the water-bearing material is not coarse enough to admit of this form of construction and is about uniform of texture, a substitute may be employed, as shown by Fig. 11. In this instance the casing with attached strainer plugged at the bottom is sunk to the impervious substratum or to an intermediate position beneath the surface of the ground. A smaller drop-pipe without per-

forations, but with an open bottom, is made to terminate a foot or so above the strainer-plug and at the other end to connect with the siphon or suction-pipe. The casing remains open or loosely covered at the top in a manner to admit free passage of air into and out of the well-casing. The danger of an air leak is entirely removed by this arrangement no matter how much the water-table may be depressed below the top of the strainer, provided the bottom of the drop-pipe retains its water seal. It seems

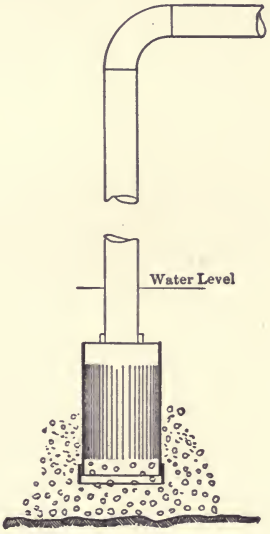


FIG. 10.

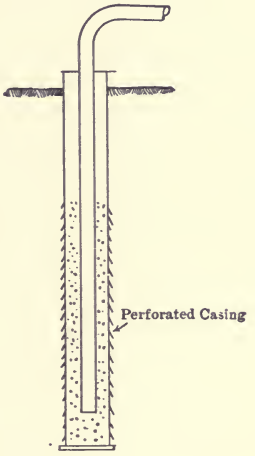


FIG. 11.

unnecessary to state here, considering previous pages of this chapter, that the strainer in this form of well should never exceed one-half the depth of the water-bearing material, and that the slots of the strainer should be as coarse as it is possible to use in any given water-bearing material even though it may be necessary to subject the well to a thorough preliminary washing to remove all the fine sand about the strainer before the well is put in commission. In fact the washing process is in most instances a necessary part of the construction work with any form of strainer fitted for use in a public water-supply, except in cases where the sand is decidedly fine and a very small yield of water only is expected. It is always better to have a strainer too long than too

short, and whenever a doubt arises as to the danger of air leaks through exposed strainer perforations it is better to adopt the open-well construction shown in Fig. 11 rather than to shorten the strainer unduly.

Drop-pipes. The drop-pipe connects the well with the siphon or suction-pipe by two forms of construction in the usual practice. The one form consists of a lateral extension of the well-casing to the siphon or suction-pipe, making the casing a part and parcel of the system exposed to vacuum conditions and practically a closed-well system of construction; in the other form of construction, the open-well construction, the drop-pipe descends inside the well-casing and is no part thereof. Figs. 9, 10, and 11 show the two forms of construction referred to. In the latter form of construction the vacuum affects the drop-pipe only, the inside of the well being always exposed to atmospheric pressure. The well itself with the same strainer capacity will yield practically the same amount of water under both forms of construction; accordingly the choice of the method of construction becomes technically a matter of adaptability to the physical conditions surrounding the water-bearing material.

First cost is always an important factor of consideration. Where the water-bearing material is deep and where the most desirable substratum for the strainer-bed lies below the influence of the vacuum effect of the siphon pipe the relative cost of the two methods of construction described may be of primary consideration; but, on the other hand, where the range of vacuum effect extends to a level below that of the upper portion of the strainer, the danger of the admission of air through the exposed openings of the strainer renders the question of first cost subordinate to questions of a purely technical character relating to the mechanical superiority of the one form over the other form of construction, having in view the security of a continuous service. Whenever the plan of water-supply development embraces a series of wells connected with a siphon or suction pipe it is perfectly apparent that all wells must be equally responsive to vacuum effect if under equally favorable conditions of mechanical make-up each well is expected to deliver its proportionate part of the total yield of water. In order that this condition of yield may prevail

throughout the system, the length and diameter of drop-pipe must be proportioned for equal resistance of a unit yield of water. Just what the measure of this resistance should be is a matter for local determination, but as a rule it should be low, perhaps not more than 1 or 2 feet of total loss in any drop-pipe.

The drop-pipe should rise continuously to the siphon pipe in order to avoid any possibility of an air-trap, and should be supplied with a stop-valve. Sometimes a check is also added, the object of which is to hold the water in the siphon pipe whenever the siphon is not in operation.

Siphon Pipe. The line of pipe connecting the system of wells is termed a siphon pipe when it dips into an open well from which the pumps draw their supply of water, and receives the name of suction-pipe when it connects the system of wells directly with the pumps. In either case the discharge head or pressure producing the flow of water from the wells depends upon the vacuum formed by the pumps directly in the suction-pipe, or indirectly in the siphon pipe through the medium of the open well. Preference is usually given to the siphon connection with the wells particularly when a steady flow under a high vacuum is desired, for the reason that the water in the suction-well feels the pulsation of the pumps moderately only, and the pump effect is communicated to the tubular wells to a degree depending inversely upon the size of the suction well into which the siphon discharges and from which the pumps take the suction water.

The pipe line, whether acting as a siphon or suction pipe, should be laid, when practicable, with a continuous and slightly ascending grade either toward the well or the pump, as the case may be. No form of obstruction should hinder the flow of air and gas from the furthermost water-supply well to a summit from which the air can be taken out mechanically. Comment is scarcely necessary to emphasize the care with which the grade should be established and the pipe jointed into a straight and graded line, but a word of caution is necessary judging from the apparently careless manner in which some work of this character has been done. Flange-jointed cast-iron pipe is the best material for work of this kind, but the expense its use entails is often prohibitive. The best substitute is lead-

jointed cast-iron pipe with a slab of reinforced concrete under each joint. The slab of concrete should be formed in the bottom of the trench to conform nicely to the proper grade for each joint and each special casting, and after the pipe is laid and jointed it may be found an act of prudence well repaid to rigidly support each pipe where it enters the bell of its neighbor as a provision against settlement or straining of a joint when the earth removed from the trench is returned.

It is necessary to provide a chamber of considerable size at the point where the siphon descends into the suction well. An air-pipe should lead from the top of this chamber to a dry vacuum-pump located in a convenient place, preferably in the engine-room, with which to exhaust the siphon of air and gas accumulating during the operation of the siphon. The down-take pipe should descend to a level in the suction-well twenty-five or more feet below the level of the pump. When laid out in this manner the siphon line should be found, and if the work of design and construction is carefully done, will be found as positive and reliable of operation as any form of construction that can be devised.

The substitution of a suction-pipe for a siphon is scarcely warranted from a purely technical point of view, although sometimes advisable in connection with a small water-works or when

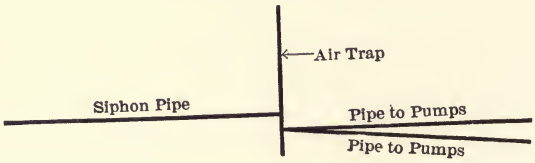


FIG. 12.

the local substrata are of such a character as to entail unusually heavy expense in the construction of a suction well deep enough to form the requisite vacuum. In such instances a vacuum chamber near the pumps is necessary and sometimes in addition to the chamber an air-trap in the suction-pipe similar to that of Fig. 12.

The pipe entering the trap from the wells is a few inches higher than the pipe leaving the trap to connect with the pumps,

The rising pipe of the trap should be large enough to admit of easy circulation of water in dropping from the inlet to the outlet, and at the top should have an air-pipe connected with a dry vacuum-pump. If the trap is large the effect of pump pulsation upon the well may be greatly softened.

Usually the line of water-supply development is restricted to a direction normal to the slope of the water-table. Therefore the requirements relating to future extension should be given due consideration during the original design and construction. Accordingly it may not always be the part of economy to reduce the diameter of a siphon or suction pipe as it recedes from the station and finally to end it in a pipe of a capacity for the flow of but one or two wells. This course of procedure, though common with small water-works, is injudicious, for it defeats all aims at economy when the occasion arises for an extension. The velocity head and frictional resistance of flowing water in either a siphon or suction pipe, as well as in the drop-pipes of the wells and through the well-strainers, are proportional to the vacuum up to the practical limit of 20 to 25 feet. If for any reason the line of pipe connecting the system of wells is too long or of insufficient size to admit of the full flow of the wells without increasing frictional resistance beyond the supporting power of the vacuum the system becomes throttled in the proportion that the resistance of full flow exceeds the available vacuum. Consequently a constructed system of piping connecting a series of wells which in the outstart absorbs the head produced by an available vacuum will gain little, if any, by extensions of any sort. Therefore a liberal allowance of pipe capacity is essential even to the extent of a uniform diameter of siphon or suction pipe in anticipation simply of future extensions.

Perhaps a diagram may make the points more evident and impressive.

In sketch, Fig. 13, is illustrated a siphon connecting a series of wells discharging into a suction well, also the suction pipe from the pumps entering the suction well. The natural ground-water table is shown by the higher of the two broken lines and the vacuum line, say 20 feet below the center of the pump-plungers, by the lower broken line. The air having been exhausted

from the siphon pipe and siphon conditions having been established therein, the head of water under which the siphon works is represented by the difference of level between the water in the suction-well as it is depressed by the operation of the pumps and the water in the ground at natural ground-water level or the head $BC+CD$. This head becomes a maximum at the lowest level that the pumping-engine is capable of maintaining the water in the well. The head AB represents the lift of water produced by the vacuum conditions from the natural level of the water-table to the highest point of the siphon pipe and reduces to zero when the siphon pipe is laid at the level of the natural ground-water table. This head AB has no bearing upon the flow of water

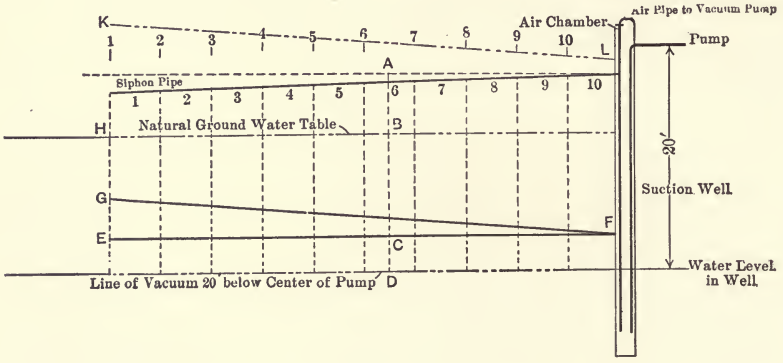


FIG. 13.

through the siphon, for when the water in the well rises to the level of the ground-water, flow through the siphon ceases and the head AB, which is a negative head, simply holds the water in the pipe system, that is to say, keeps the system primed. The amount of this negative head or lift that is permissible depends upon the practical limit of a siphonic vacuum, 20 to 25 feet, and upon the amount that it is desirable to depress the natural ground-water level. From the standpoint of economy of construction, it may have some plus value with relation to the ground-water table in order that the work of excavation may be "in the dry."

The head BD between the ground-water table and the vacuum line is made up of two parts, the part BC representing the distance between the normal water-table and the depressed water-table at the wells, or the head consumed in producing the flow of

water through the sand towards the system of wells, or the value i in the general formula $v=ki$; the other part CD is the head consumed in overcoming the resistance of the strainer and the entire pipe system. Now since the head BC is the effective head which delivers the water to the wells through a highly resisting material, the sand, it follows that it should be impaired in the operation as little as possible by frictional resistance through the wells and pipes of the water-supply system, or, to state the case diagrammatically, the flow head BC should be large with respect to the pipe friction head DC.

Certain conditions of flow may be assumed in order to illustrate in the diagram the relation of flow head to friction head. Let a siphon pipe be 1000 feet long, 12 inches in diameter from the most remote well to the suction well, connected at each 100 feet of its length with a tubular well, each well furnishing 100,000 gallons of water a day. It may also be assumed that the yielding capacity of the sand has been the basis of the above estimate of yield per well and that the well and its strainer and drop-pipe have been proportioned for a frictional resistance of 3 feet.

The frictional resistance in the siphon changes with the addition of a well at every 100 feet of its length and is cumulative, that is to say, that the resistance in section 10, including the drop-pipe, is due to the flow of water from all the wells; the resistance head at well 9 is the resistance at well 10 plus the resistance of a flow of 900,000 gallons per 24 hours through 100 feet of pipe between wells 9 and 10, the resistance at wells 8 is that at 9 plus the resistance of a flow of approximately 800,000 gallons through its section of 100 feet of pipe, etc. We compute the resistance head at well 10, including strainer and drop-pipe, at 3.2 feet and that at well 1 at 3.6 feet, and a velocity varying from 2 feet at well 10 to about 0.2 of a foot per second at well 1.

Now suppose it were desired to double the supply of water by extending the 12-inch pipe 1000 feet and by adding 10 additional wells, making the total length of the water-supply line 2000 feet with 20 wells attached. We would then have an assumed discharge of 2,000,000 gallons at original well 10, and 1,000,000 gallons at original well 1, and a corresponding resistance line GF much steeper than the former line EF and showing a

resistance head at well 10 of 3.7 feet, while at well 1 it is 7.5, with a velocity of flow varying from 2 feet at well 1 to nearly 4 feet at well 10.

The heavy resistance head at well 1 resulting from an effort to put an additional 1,000,000 gallons of water through the original 1000 feet of siphon pipe extends its effect throughout the extension and leaves a very much reduced head, *HG*, to influence the velocity of approach of the ground-water to the wells.

The hydraulic conditions will not manifest themselves precisely as outlined, because in the actual adjustment of the hydraulic conditions the true hydraulic slope line will lie somewhere between the line *EF* and *GF* corresponding to some intermediate rate of discharge from the 20 wells on the 2000 feet of siphon pipe considerably below the 2,000,000 gallons rate used as the basis of this estimate, the case being somewhat exaggerated to show the error into which one may fall if due consideration is not given the question of probable future requirements in the original design and construction. Had the future requirements been anticipated in the first illustration, the size of siphon pipe would have been fixed larger than 12 inches.

An effort to increase the flow of the entire system of wells, by independently increasing the vacuum on the siphon, would create conflicting siphonic conditions whenever that vacuum line falls below the vacuum line indicated by the level of the water in the well, for then the direction of siphon-action would be reversed from the point where the independently-made vacuum line falls below the water-level of the suction-well, and accordingly there would be a tendency towards flow from the suction-well into the ground. Moreover, this siphon pipe should be large enough that a line *KL* drawn parallel with the ground-water hydraulic grade line of the highest possible rate of draft, from a point *K*, 25 feet above the vacuum line, shall be above the siphon at all points, otherwise the siphon will flow only partially full or with a pulsating flow through that portion above the line *KL*.

There is no necessity of setting the pumps or the siphon pipe lower than is necessary to produce a vacuum which insures an average depression of the ground-water table lower than one-half the depth of water-bearing stratum.

In order to guard against all conflicting conditions it is necessary to thoroughly study the local conditions and to start the original construction with due regard to probable future requirements and a harmonious arrangement of the various parts of the water-supply system. One cannot venture to place precise limits upon the velocity of flow and the loss of head due to pipe resistance in a siphon pipe without definite information regarding local conditions and requirements. However, an endeavor has been made to make clear the general requirement that both of these elements should be given a consistent minimum in any well-water-supply design where the depth of water-bearing material is small and accordingly where every foot of head of ground-water down to the level producing the condition of maximum flow should be utilized advantageously. For let it be remembered that every foot of head used to overcome resistance of flow in pipes is just so much head diverted from velocity effect upon the ground-water approaching the wells through the surrounding sand-bed. Moreover it is always desirable to make the draft from each well of the system of wells as nearly uniform and equal as practicable, which is an impossible result with a gang of wells operated under high suction or siphon pipe-resistance; for in this case the rapidly ascending hydraulic gradient of the ground-water parallel with the gang of wells results in a rapid and progressive decrease of yield of the individual wells in a direction away from the suction-well or the pumps, and finally reaches a point where the hydraulic grade line approaching nearly to the normal level of the water-table admits of no flow or only a pulsating flow of remote wells.

Returning to a consideration of a tubular-well and siphon system suited to the condition of water-bearing stratum illustrated by Fig. 2, on page 51, where

$h = 20$ feet, the depth of water-bearing material,
$k = 1000$ for a No. 36 sand,
$l = 500$ feet,

the maximum discharge of the direct current of approach of water through the sand is to be found by substitution in the formula

$Q = \frac{1}{4}\frac{kh^2}{l} = 200$ cu. ft. per day per lin. ft. and double this amount, or 400 cu. ft. per day, for both sides of a line of wells.

The average maximum discharge of a tubular well under the conditions stated may be approximated by formula (15).

$$Q_{max.} = 0.0475 \times kh^2 = 19{,}000 \text{ cu. ft. per day.}$$

The distribution of the wells may be approximated by dividing 19,000 cu. ft., the capacity of one well, by 400 cu. ft., the discharge per lineal foot of water-bearing material under a head of $\frac{h}{2} = 10$ feet, and amounts to 47.5 feet.

Having no analysis of the water-bearing material, assume it to be uniformly a No. 36 sand, the size previously computed. Observations of the capacity of an ordinary slotted brass strainer with slots $1\frac{1}{4}$ inches long by 1/32 of an inch wide and 13 per cent of the total metal surface indicate a capacity of about 12,000 gallons or 1600 cu. ft. per day per lineal foot in fine sand. Accordingly a strainer of this rated capacity should be about 12 feet long to absorb 19,000 cu. ft. of water per day. But as the limit of length is 10 feet the deficiency must be made up by increasing the slot area or efficiency above that of the rated strainer.

The velocity of radial flow at the exterior of the strainer is $\frac{19000}{10\pi d}$, or 1210 feet per day for a strainer 6 inches in diameter.

This diameter tested by formula (16) became

$$d = \frac{0.03 \times 1000 \times 20}{1210} = 0.495 \text{ of a foot.}$$

The diameter of a drop-pipe which can discharge 19,000 cu. ft. or 142,500 gallons per day with a frictional loss of 1 foot in 25 feet, including valves and fittings, is about 3 inches. The requisite diameter of drop-pipe is so much less than the diameter of the well that the open-well plan of development naturally suggests itself. Of course the drop-pipe should be as short as

possible, but as it must descend to within a foot or two of the bottom of the well the vertical portion must approximate 15 feet and the horizontal portion between the well and siphon should be 10 feet or a little less.

Practically the wells should be distributed alternately on either side of but near the siphon pipe, about 48 feet apart. Mutual interference among the wells so arranged must necessarily result, but as they are designed of a capacity to absorb the direct maximum flow of water from both sides of the alignment, the question of mutual interference becomes a matter of little or no importance.

If the average draft from the wells is 3,500,000 gallons per day and the rate of emergency draft is 5,000,000 gallons per day, then under the conditions of the problem the number of wells becomes $5{,}000{,}000 \div 142{,}500 = 35$, and the total length of the line of development 1680 feet.

The siphon pipe should be large enough for the stated maximum flow plus a reasonable allowance of capacity to admit of future extension, but as there is no adequate data limiting the supplying capacity of this particular valley it may be assumed for the purpose of this illustration that through an extension of the wells the supply may be increased about 50 per cent, or to about 7,500,000 gallons. Accordingly the siphon capacity should be designed for a total length of about 2500 feet with a capacity of 7,500,000 gallons at or near the suction-well and proportionately less capacity as the pipe recedes for that well. Inasmuch as it is desired to make the greatest use of the ground storage and to insure as high a yield as possible of each well, a low friction head should be maintained throughout; in fact, for so thin a water-bearing material, the siphon-pipe resistance should scarcely exceed a rate of 2 feet per thousand for the average draft, or 4 feet per thousand for maximum draft. Accordingly the diameters and lengths of the siphon pipe for an average draft of 5,000,000 gallons per day at the suction-well may be 1000′ of 24″, 500′ of 20″, 500′ of 16″, 250′ of 12″, and 250′ of 10″ pipe, offering a total resistance of about 1.6 feet for the average rate of draft and 3.6 feet for the maximum rate of draft.

The stated continuous yield of 5,000,000 gallons of water

from 2500 feet of tubular-well development depends upon the extent to which it can be sustained by the rainfall together with the substantial assistance of river reinforcement when necessary, and embraces a discharge of 2000 gallons from two sides or 1000 gallons from one side per lineal foot of the line of development. It requires an average velocity of approach of over 6 feet per day in terms of a solid column of water at a grade of about 24 feet per 1000 feet and a catchment area extending over $6\frac{1}{2}$ miles back from the line of development absorbing 17 inches of the annual rainfall, or heavy local river-reinforcement.

Where conditions confine the support of the yield largely to one side of the line of development, the foregoing measure of velocity, grade, and area of catchment territory must be proportionately greater or the yield must be better sustained by river reinforcement.

For a second illustration consider the Muscatine conditions, where the saturated bed of gravel is 35 feet deep of an average effective size of sand of No. 18, corresponding to a value of k equal to 4000, but underlaid with 10 feet of No. 13.5 gravel, corresponding to a value of k equal to 8337. The greatest permissible depression of the ground-water table by pumping is 10 feet and the corresponding value of discharge for average porosity of the sand by formula 13 becomes

$$Q = 0.19kx(h-x) = 0.19,\ 4000,\ 25(35-25) = 190{,}000 \text{ cu. ft.}$$

per day, equivalent to a velocity of radial flow at the circumference of a 6-inch well of $\frac{190000}{3.142,\ .5,\ 25} = 4840$ feet per day. But as it is desired to confine the strainer to the 10 feet of coarse material and at the same time take advantage of the full yield of water for the 10 feet of available head, the requisite velocity of radial flow becomes $\frac{8337}{4000} \times 4840 = 10{,}088$ feet per day. Accordingl the strainer must have a diameter which admits of the cutting of a grade of slots which shall have sufficient hydraulic efficiency to absorb water at the rate of 190,000 cu. ft. per day.

Applying formula (17) we find for the grade of sand

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represented by value $k=8337$ the requisite area of strainer openings to be

$$bd = 0.00065, 8337, 25\sqrt{35-25} = 428 \text{ sq. in.}$$

Thus in very coarse material like that at Muscatine a single well with a properly proportioned strainer can be expected to furnish an exceedingly large volume of water.

The layout of a well system may be proportioned as to the length of development by the diagram shown on page 47, while the strainer capacity and pipe resistance should be proportioned for the emergency yield when a draft upon storage is generally necessary.

Probably no difficulty will be experienced in comparing the costs of the gallery and the tubular-well methods of water-supply development upon the basis outlined, and it is hoped that the approximate formulæ may prove of assistance not only in making this comparison, but also in the design of ground-water supplies where large yields are required.

The proximity of a town to a desirable locality for the development of a ground-water supply may sometimes be a source of apprehension and is often considered a condition calling for a thorough sanitary investigation of the underflow. If the desirable locality is above the town, that is to say up-stream with regard to the direction of the slope of the underflow, an investigation is unnecessary for the reason that the pollution which might enter the underflow from the town would naturally follow the drift of the underground water. Undoubtedly a location above the town of a ground-water supply is a safe precaution, but the danger of pollution of a similar water-supply located below the town may be often overestimated and exaggerated. For instance, the valleys of such rivers as the Mississippi and Missouri possess a slope in the direction of flow of the rivers practically the same as the rivers themselves and scarcely exceeding in the two valleys referred to and similar ones 6 to 12 inches per mile, or from 0.095 to 0.19 of a foot per thousand feet.

This exceedingly small slope can scarcely produce a velocity of ground-water flow in the direction of the valley greater than four- or five-tenths of a foot per day, and is rendered almost ineffective by reason of the controlling effect of the slope approximately normal to the general direction of the river, which usually exceeds the former tenfold or more. The influence of flow riverward so far predominates that the underdrainage from the town can progress but slightly in a direction parallel with the river. The drive of ground-water inland during a rise of the river can scarcely modify the general direction of flow riverward, because when the crest of the flood-wave has passed and the river recedes rapidly the resulting increased ground-water slope accelerates the underflow in the direction of the river.

A local drought may flatten the ground-water table somewhat, but scarcely to the extent of giving the balance of power to a parallel tendency of flow. Should the drought become protracted and extensive its influence progressively reduces the discharge of the river and the resulting falling stage maintains the normal slope of the water-table as the river draws upon the underground storage for its support.

It is conceivable, however, that in valleys of heavy slope the parallel flow may at times predominate. In that event the element of time becomes a factor and the interval which must elapse between the introduction of pollution from town drainage and the arrival of the water at the site of the water-supply may be fully sufficient to remove everything but the evidence of past pollution. A study of local conditions should in any case determine whether apprehensions of pollution have any substantial support.

Generally the danger of serious pollution of the character referred to is to be guarded against chiefly in the immediate neighborhood of the water-supply works, and if proper precautions are here observed to prevent deposits of polluting matter underground the really dangerous element of underground-water pollution will have largely disappeared.

Removal of Iron.

Iron in considerable amount in a ground-water imparts to it an inky taste, a disagreeable odor when accompanied, as it often is, by sulphuretted or carborated hydrogen gas, and upon standing a repulsive yellowish appearance. A water containing iron is colorless and transparent when freshly drawn from the ground, and to all appearances as inviting as any spring-water. The slightly disagreeable taste ceases to repel upon acquaintance, particularly under the impression that this class of water possesses medicinal properties. It is only upon standing that its attractiveness as a beverage, either for medicinal or natural purposes, ceases, for the partially oxidized iron held in solution by carbonic acid quickly absorbs oxygen from the atmosphere, becomes disunited from the carbonic acid, and assumes the state of an insoluble mineral sediment of a yellowish color. The discolored water clears as the iron sediment gradually settles, but not without a tell-tale oily appearing film upon the surface of the clarified water. Continued exposure darkens the sediment to a deep rusty shade which adheres very firmly to any vessel repeatedly exposed to the water.

This peculiarity of the iron to discolor utensils of various kinds renders it particularly objectionable for domestic uses, and to even a greater degree in cooking because of the offensively rusty color it imparts to many vegetables and the deep inky shade it dyes tea and coffee.

While the presence of iron in a ground-water in sufficient amount to precipitate upon exposure to air is readily detected visually by one acquainted with its character, still there are instances where its presence and general characteristics became a complete revelation to communities which have preferred ground-water to other sources of public water-supply. They either never suspected its presence in the selection of the source of supply or were wholly uninformed as to the many annoyances which always follow its use for general water-supply purposes. Association with it as a public water-supply never reconciles one to its use. It is regarded in sort of a spirit of toleration as a thing to be endured for a time, mingled with the ever-present hope that a

remedy can be none too speedy. The rusty-colored liquid which marks the morning flushing of a service pipe or is periodically ejected from a hydrant on the dead end of a street pipe does not inspire one with much faith in the medicinal qualities of a chalybeate water whatever the facts may be with reference to the water freshly drawn from the ground; in fact there is no association with it about the household that can inspire much respect or promote much friendship for it.

The remedy for iron in water has many suggestions in nature. Wherever spring-water containing iron emerges from the ground a rusty deposit follows the course of the rivulet for a considerable distance from its source. The oxide of iron, adhering firmly to almost any obstruction with which the mineralized water comes in contact, is the result of the aeration which the water naturally receives during the first brief period of its surface flow. This natural process of oxidation by aeration can be much hastened by spraying the water in any manner which will expose it thoroughly to the atmosphere. It has been successfully practiced abroad for many years and to a more limited extent and for a briefer period of time in this country.

The attractive part of the process, although purely a chemical process, is that it is altogether natural and accordingly requires the use of no chemical to promote chemical action. The air supplies the chemical that is needed to oxidize the iron and reduce it to an insoluble condition. Another attractive feature is that the process may be applied very cheaply and equally successful on a large scale as well as on a small scale.

After the iron is precipitated as an insoluble oxide the problem of straining or filtering out of the water the iron sediment is a problem in many respects similar to the clarification of a slightly muddy surface-water. As much as 90 per cent of the iron may be removed by sedimentation practiced under very favorable conditions, perhaps more favorable than is attainable in operating a water-works. The process of sedimentation, however, is too uncertain to apply to the removal of iron except as a preliminary process intended to precede and to lessen the work of filtration.

It is not unusual to find iron present in ground-water to the

extent of one-half to one grain per gallon, but in one instance the amount was observed to reach the high figure of over four grains to the gallon, an abnormal condition, however, produced possibly by the existence for many years of stock-yards upon the ground under which the sample tested was extracted.

The iron in aerated ground-water may be removed by natural filtration through comparatively coarse sand at a rate much in excess of that ordinarily employed in slow sand filters for the purification of water polluted with sewage.

The depth to which the iron penetrates the filter sand depends altogether upon the rate of filtration. At a rate of 300 to 400 gallons per square foot per day it is found that the removal of a layer of about three-quarters of an inch of sand at a cleaning will restore the effectiveness of the filter at least to an extent that with progressive scrapings of the stated amount the filter renders effective service until the sand-bed reaches the practical minimum of thickness before the sand removed at the various scrapings is returned to its original place in the filter. The three-quarters of an inch does not represent the extent of the penetration of iron into the filter sand, as it is found at a depth of 6 inches, but the clogging effect is confined to practically the upper three-quarters of an inch and chiefly in the film of iron which covers the surface of the filter sand.

The ordinary rate of filtration employed in mechanical filters is fully seven times the rate above mentioned, or about 2000 to 2800 gallons per square foot per day, and would carry the iron deep into the sand, and at times carry it entirely through the filter into the filtrate unless a coagulant be used. Although iron itself is a coagulant when employed to remove ordinary sediment from water, still when iron itself becomes the sediment it is sought to remove, the process is then akin to the removal of hydrate of alumina from a water treated with a solution of alum.

A little over a year ago a water-purification works was installed at Richmond, Missouri, which so successfully removed the iron from the water that a description may prove interesting.

The source of supply is from four 8-inch wells, about 25 feet apart, of an aggregate capacity of 800,000 gallons of water per twenty-four hours. The well-strainers are 16 feet long, resting

upon the bed-rock underlying the Missouri River valley and surrounded by a stratum of coarse sand 21 feet thick overlaying the rock. The site of the water-supply is about four miles from the town and is the only accessible site promising to furnish an unfailing supply of water.

Although the presence of iron in the ground-water was observed at the time the works were constructed over seven years ago, no funds were then available for a purification works and accordingly no effort was made at the time to introduce them. It was remarked at the time of construction that the iron might diminish with the protracted and continuous use of the water, but after six years of water service the iron increased rather than diminished, and at the time of the construction of the purification works the iron amounted to about $\frac{3}{4}$ of a grain to the gallon or about 12 parts per million. During the six years of use of the iron water the annoyance gradually increased and popular objection intensified accordingly.

The results of the operation of an experimental plant erected and operated for the purpose of demonstrating the feasibility of removing the iron led to the following conclusions:

1. Aeration completely precipitates the iron and reduces it to a condition in which it can be partially removed by subsidence and thoroughly removed by filtration.

2. Subsidence alone can remove but about 80 per cent of the iron under favorable conditions in a subsiding basin of a capacity one and one-half or more times greater than the maximum daily water consumption.

3. Filtration of the aerated water removes practically all the iron, that is to say, reduces it to 0.05 part per million.

4. The amount of iron in the natural water is so large that preliminary subsidence after aeration of the water will reduce the work of the filter and prolong its life between scrapings.

5. By combining in one process aeration, subsidence, and filtration an attractive water can be furnished which will be found as pure and wholesome as the best spring-water.

The purification plant as constructed consists of three concentric rings of concrete. The inner ring is 17 feet, the middle ring 38 feet, and the outer ring 60 feet in diameter, as

shown by the sectional elevation, Fig. 14. The rings are respectively 6 inches, 11 inches, and 18 inches thick. The annular space between the concentric rings is 10 feet, the depth 6½ feet in the subsiding and aerating basins, 11½ feet in the filter, and 13 feet in the clear-water basin. The space inside the inner curb is the aerating basin connecting, by means of two notches 18 inches wide and 12 inches deep, with the annular space between the inner and middle curbs which forms the subsiding basin. The annular space between the middle and outer rings is divided by transverse walls into three divisions; one-half the annular space is the clear-water basin and the other half, divided equally by a third division wall, forms the two filter receptacles. There is a regulating well at each of the

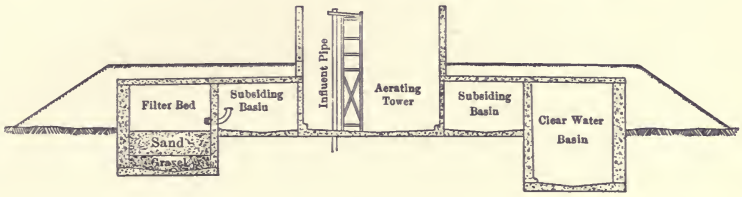

FIG. 14.

two division walls separating the filters from the clear-water basins, which receives the water from the filter underdrains and delivers it into the clear-water basin through valves set at different elevations and attached to pipes passing through the division walls. By means of these valves the rate of flow from the filter can be controlled within desirable limits.

The clean-out opening for each filter consists of a 14-inch sluice gate set in the outside curb and housed with concrete side walls and cover extending to the slope of the earth embankment.

The subsiding basin connects with the filter by a 6-inch pipe with attached stop-valve, which is carried upward inside the subsiding basin to a level a few inches below the roof. An overflow pipe connecting with the subsiding basin limits the level of the water.

The drain-pipe connecting with both the subsiding basin and the aerating basin has a stop-valve outside the embankment and a disc valve inside the aerating basin.

A 6-inch pipe delivers the water from the well pump into the aerator. The aerator consists of four trays, each 2 feet by 6 feet in the clear and 6 inches deep attached at two feet intervals to four posts set upright on the bottom of the aerator basin. Each of the four pans has a bottom of No. 8 wire net with $\frac{3}{4}$-inch mesh supporting a galvanized iron lining peforated with $\frac{1}{8}$-inch holes spaced $\frac{1}{2}$ inch apart. The water from the rising main enters the uppermost tray in a sheet about 6 feet wide and showers down through the perforations of the succeeding pans into the basin below. The total drop of the water is about 8 feet.

An 8-inch suction-pipe connects the clear-water basin with the high-pressure pump which delivers the purified water to the consumers.

The bottom of the clear-water basin and filter receptacles, the center and the inner curbs and the roof are all of 1–3–5 Iola Portland-cement concrete, reinforced with the Johnson corrugated steel rods. The exterior curbing is banked with earth and the concrete roof has a covering of earth 18 to 24 inches deep.

The aerator is surrounded by a concrete curb 2 feet high and a brick curb for an additional height of 7 feet. It is open at the top.

The bottom of the filter receptacles, clear-water basin, and aerator basin were constructed in radial block alternately. The outside curb of the clear-water basin and one-half of two division walls were constructed monolithic. The middle curb, the other half of the two division walls above referred to, the regulating wells, the outside curb of the filters, and half the division wall between the two filters are constructed monolithic as shown by Plate VII, page 91. Plate VIII, page 93, shows the inner curb completed and the outer forms removed; Plate VII is also a general view of the work during construction. Plate IX, on page 95, and Plate X, on page 97, show details of construction at different stages of progress. Plate IX in particular gives an interesting view of the method of constructing the roof slabs of concrete in forms suspended from A-shaped wood frames. The entrance to the clear-water basin is where the ladder is seen protruding, the square opening is the regulating well, the A frames over the filters with

PLATE VII.

Purification Works, Richmond, Mo.

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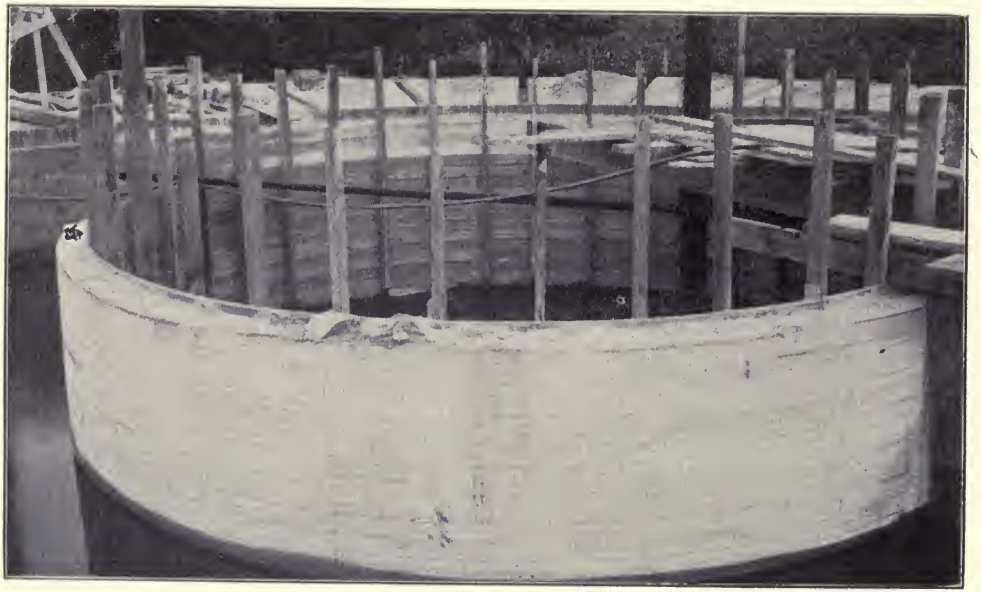
PLATE VIII.



Purification Works, Richmond, Mo.

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PLATE IX.

Purification Works, Richmond, Mo.

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attached platform are in readiness to receive the concrete and steel for a reinforced radial roof block. The roof blocks were put in alternately. The platform which had served the purpose of support for the construction of one radial roof block was loosened at the end of about a week and pushed laterally on the girders suspended by bolts to a pair of rafters, to the opening of an alternate block and thus acted as a support for a second block of concrete. The 6-inch radial channels left after the removal of the A trusses were then filled in on a plank suspended beneath the opening.

Plate XI, page 101, shows the completed structure with its earth covering.

Plate XII, on page 103, is the aerator in operation.

The concrete was of mobile consistency when put in the forms and required no ramming. A hoe with a straightened shank was used to work the mortar about the steel and to a smooth face at surface of contact with the forms. There was no occasion to retouch with a trowel the concrete faces except when 1, 3, 6 concrete was used in a portion of the outer curb. The forms were removed after an interval of thirty-six to forty-eight hours after the concrete was put in place, with the exception of the roof concrete, which was allowed one week's time of setting before removing the forms.

The earth roof cover was put on with teams and scrapers, the teams passing indiscriminately over the naked concrete slabs one month after construction.

The filter underdrain is of 10-inch split sewer-pipe laid with open joints upon the concrete bottom along the center line of each filter and communicating with the regulating well. The split pipe was covered with $2\frac{1}{2}$-inch road metal, and over this 16 inches of screened stone of a size to pass a screen with 1-inch mesh and rejected by a screen of $\frac{1}{2}$-inch mesh, including a top dressing of fine stone passing a $\frac{1}{2}$-inch mesh and rejected by a $\frac{1}{4}$-inch mesh. The filter sand was procured from a near-by bank, sorted, washed, and then deposited to a depth of $3\frac{1}{2}$ to 4 feet in a manner which avoided any tendency of stratification. The mechanical analysis of the sand is as follows.

Passing	No.	6	sieve,	100	per	cent.
"	"	10	"	97.8	"	"
"	"	14	"	93.46	"	"
"	"	18	"	87.86	"	"
"	"	20	"	75.75	"	"
"	"	40	"	14.26	"	"
"	"	60	"	2.16	"	"
"	"	80	"	0.68	"	"

The effective size is No. 47 or 0.41 mm.; the uniformity coefficient 2.12.

The first filter put in commission furnished the following record:

1st	day,	filter-head,	6	inches,	rate	367	gallons	per	sq. ft.
2d	"	"	8	"	"	367	"	"	"
3d	"	"	$9\frac{1}{2}$	"	"	367	"	"	"
4th	"	"	$11\frac{3}{4}$	"	"	367	"	"	"
5th	"	"	17	"	"	367	"	"	"
6th	"	"	19	"	"	367	"	"	"
7th	"	"	25	"	"	367	"	"	"
8th	"	"	30	"	"	367	"	"	"
12th	"	"	56	"	"	367	"	"	"

The filter was scraped on the twelfth day after starting.

A run of forty-nine days gave the following result, which substantially represents a fair rating of the filters.

Total amount of water pumped	3,885,700 gallons
" " " dirty sand removed	16.17 cu. yds.
Numbers of times the filters were cleaned	6
Gallons of water pumped for each 1 inch of sand removed	555,100
Number of cubic yards of sand removed for each 1,000,000 gallons of water filtered	4.1
Number of gallons of water filtered for each cubic yard of dirty sand removed	240,300

Within six months after the filtered water was supplied to the city, the water consumption had practically doubled.

PLATE XI.

Purification Works, Richmond, Mo.

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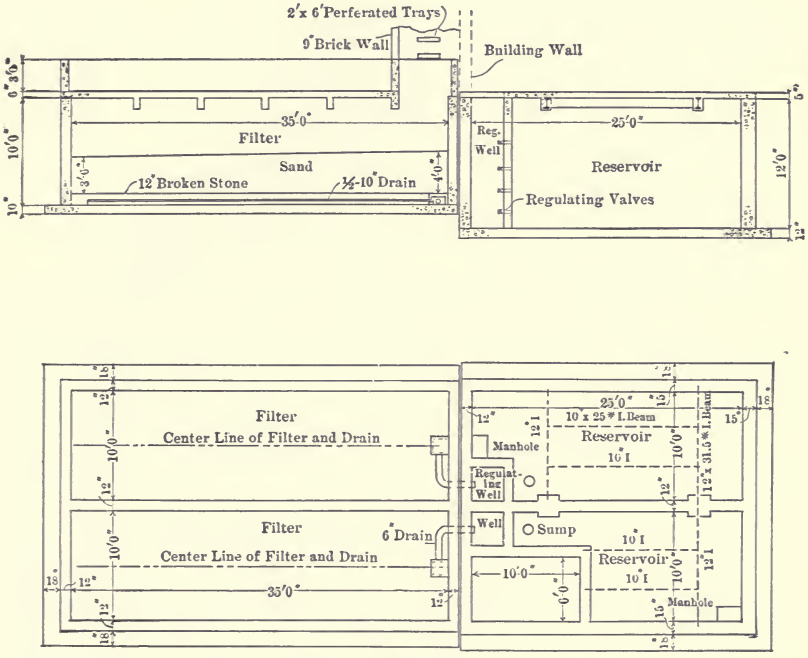
The dirty sand is removed from the filter by manual labor and is washed in a trough and stored in a bin until needed to replenish the filters.

Since this work was completed another purification works for a similar purpose has been constructed for the town of Liberty, Missouri. This purification works is more compact than the

PLATE XIII.



Plan of Purification Works at Liberty, Mo.

Richmond works for the reason that it was built at the same time as the water-supply and pumping station. It is rectangular in form and constructed of 1–3–5 Iola-Portland cement concrete reinforced with the Johnson corrugated steel rods.

Plate XIII shows the plan and the elevation of the purification works.

Plate XIV is a view looking down upon the roof of the filters and enclosing parapet walls within which dirty filter sand is washed and then stored. The tower enclosing the aerator is

shown immediately behind the men sitting on the parapet walls and back of the tower walls is the wall of the pump-room, resting upon the reinforced-concrete walls enclosing the clear-water basin.

Plate XV shows the high-pressure pumping-engine installation. The engine sets upon the roof of the clear-water basin, which is also the floor of the engine-room. Through the open doorway in the partition wall the fronts of the boilers may be seen.

The bottom is a continuous slab of concrete for both the clear-water basins and the filters, but independent of each other. The side walls and roofs are monolithic, but so formed that the concrete box composing the filters is entirely independent of that composing the clear-water basin. The several partition walls in the clear-water basin are so formed as to make two regulating wells, a pump-pit for the well pump, and two compartments for the storage of filtered water. A concrete roof covers the portion where water is stored and forms the floor of the engine-room, a stairway leads down into the pump-pit, manholes are placed over each division of the clear-water basin, and wooden covers protect the regulating wells. The side walls of the clear-water basin are the foundation walls of the engine-room part of the pumping-station. The high-pressure pump sets upon steel eye-beams molded into and made a part of the roof of the clear-water basin.

The foundation walls of the boiler-house and boilers rest upon natural earth and are not monolithic with the clear-water basin.

The filters are divided by a longitudinal wall through the center, forming two equal compartments. A concrete roof covers both filters with the exception of about 5 feet at the end of the filters next to the building. A parapet wall 3 feet high extends entirely around the covered portion of the filters and around the opening in the roof. A brick tower about 5 feet high stands upon the parapet wall surrounding the large openings into the filters. This tower encloses two sets of aerator trays, one set of four for each filter. These trays differ from those of the Richmond works only in lineal dimensions, being 3 by 4 feet. The water in showering through the trays drops about 10 feet.

Water Purification Works at Liberty, Mo.

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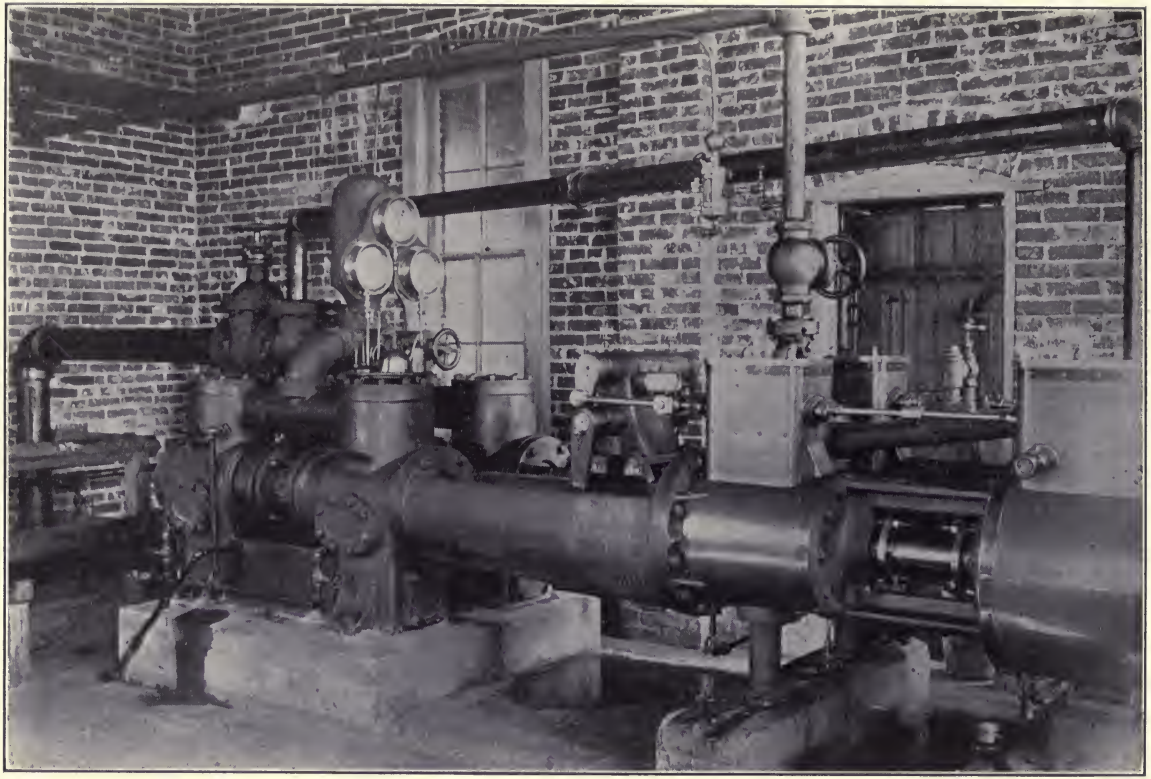

Water-works Pumping-engine at Liberty, Mo.

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The covered portion of the filter-roof enclosed by the parapet walls is for the storage and washing of the dirty sand removed from the filters. The dirty sand is skimmed from the filter by hand, but elevated onto the roof by means of a hydraulic ejector

The underdrain of the filters is composed of three parallel lines of 4-inch vitrified tile with open joints and 12 inches of graduated road metal of practically the same assorted sizes as those of the Richmond works.

The mechanical analysis of the sand, which is 4 feet deep, is as follows:

Rejected by	No.	6	sieve,	1.25	per	cent.
Passed	"	"	6	"	98.75	"	"
"	"	"	10	"	95.42	"	"
"	"	"	14	"	88.96	"	"
"	"	"	18	"	84.90	"	"
"	"	"	20	"	60.00	"	"
"	"	"	40	"	40.54	"	"
"	"	"	60	"	8.35	"	"
"	"	"	80	"	0.49	"	"

The effective size is No. 59.

Uniformity coefficient is 2.95.

All the valves for the operation of the purification works as well as of the pumping machinery, of which there are 15, are within easy reach of the engineer from the engine-room floor; the only outside valves are the two ejector valves beneath the roof of the filters. The boilers face the engine-room and can be continuously under the eye of the engineer.

Both the Richmond and the Liberty water-supply works are operated by one attendant except at the time of scraping the filters, when the attendant of the Richmond works requires a helper because the lack of facilities for mechanically removing the sand and for storing it on the roof of the filters. These have been supplied in the Liberty works. The subsiding basin of the Richmond works admits of water softening.

The entire process by which the water of both works is purified is entirely a natural process and is so simple that the ordinary water-works attendant has no difficulty in conducting all the

operations successfully without the aid of skilled assistance except for a few days when the purification works are first started, and then only to teach the attendant the principles and the few mechanical restrictions under which the purification works should be operated.

The hygienic quality of the water which is the product of such purification works as the two just described is of the highest rank. The aeration which the ground-water receives not only oxidizes the iron but also substitutes air for the gases so often present in an iron water. Filtration imparts to it a clearness which is always delightful in a drinking-water and a perfect freedom from the inky taste of the fresh well-water. In fact there is so complete a transformation of the hygienic qualities of the water in passing from the well through the aerator and filter to the clear-water basin that, so far as one may base a judgment upon taste and appearance, they are waters of totally different characteristics, and the same statement may be made of the water with regard to many domestic uses. The method of purifying the water leaves no room for the suspicion of the presence of unpleasant or undesirable by-products of chemical treatment which so often attaches to the water from filters or settling-basins which perforce must use a coagulant to remove the sediment from surface-water. In the one instance the chemical action resulting in the oxidation of the iron is altogether natural and diminishes the amount of both mineral and organic matter present in the fresh well-water, while in the other instance the use of the coagulant adds to the dissolved mineral of the raw water and also sometimes leaves suspended in a clarified river-water a portion of the coagulant in a hydrated form.

No possible exception can be taken to the bacterial purity of filtered iron water, for could any suspicion whatever attach to the well-water naturally filtered, the suspicion is entirely eliminated by the second process of filtration in covered filters and the subsequent storage of the doubly filtered water in a covered storage basin. No artificially filtered surface-water can surpass the doubly filtered well-water in hygienic or bacterial purity. Moreover, the filtered iron water approaches more nearly the popular conception of a desirable drinking-water than any puri-

fied surface-water can approach such a quality, and therefore, in small towns particularly, is the more readily accepted as a substitute for the neighborhood spring, the private well, or even the cistern, while in large towns its advent as a hygienic water would be hailed with an expression of the most genuine approval and indorsement from every side. With large towns the only questions to be considered in connection with the development and purification of a ground-water impregnated with iron are the ones of quantity and adaptability to mechanical uses. The quantity is unquestionably large in such valleys as those of the Missouri, Mississippi, Platte, and Kansas Rivers, where there is unquestionably river reinforcement to the rainfall feed of the underlying gravel-beds. But even though sufficient water may not be obtained for a full supply of water of a large city which must otherwise depend upon purified river-water for its supply, a partial supply of iron water aerated and then mixed with that portion of the supply derived from the river will through the coagulating property of the iron aid in the removal of sediments as has been pointed out in the paragraph Natural Coagulation of the chapter entitled River-water Supply. Sometimes the mixture of different waters supplies food for the natural microscopic life contained therein, and accordingly aquatic life may develop luxuriantly and upon dying, after the consumption of the food-supply, give to the waters an offensive odor and taste. This trouble need not be feared when the mixed waters are stored in the small storage basins of purification works, for there is not sufficient time for its development.

Ground-water is evidently more desirable hygienically than impure surface-water which requires clarification and purification, whether it is usable directly as removed from the ground or requires previous treatment to remove objectionable minerals; besides when ground-water must be treated, the works and plant required for this treatment are always more condensed, less expensive, and require less skilled supervision in operation than the purification works needed for the purification of muddy and impure surface-water. It frequently pays a city to go a long distance to secure that sort of water-supply rather than to install purification works to cleanse a nearer but impure surface-water. The

natural facilities in this direction are not confined to the requirements of small cities and towns, but by proper forms of development they may be made available for cities of considerable size.

It will certainly pay any city to make thorough investigation of the availability of ground-water in its vicinity before grappling with the highly expensive problem of surface-water purification, for expensive construction and maintenance must precede successful and satisfactory operation of purification works.

A zeal to stimulate the growth of high ideals as to what constitutes a standard of hygienic purity and to give structural expression to these high ideals should not lead one to such extreme lengths as to claim for the purification works which depend upon ceaseless vigil for efficiency of action the loftiest structural expression of such ideals, but rather influence one to accept such works as desirable and necessary only in the absence of means and facilities of obtaining structural expression in a simpler, more direct, and less expensive form, wherein skilled surveillance is reduced to a minimum because of the fact that the source from which the water-supply is derived is either unpolluted or so surrounded by natural safeguards as to render artificial pollution a remote probability or even a bare possibility. All energetic efforts towards progress tend to develop a few extremists either in fact because of their extremely optimistic or visionary views, or so called because their energy and foresight keep them well in advance of the general procession. A switch to by-roads from the general direction of progress, reaction, retrograde movement, a return to the old but newly paved and better-lighted route, a renewed advance are the accompaniments of general progress in technical as well as in other everyday pursuits.

Thus upon the opportunities of a community to learn of and appreciate progress, upon its means as well as upon its needs, depends very much the perfection of water-supply development in individual cases. Often simple though fairly effective methods of water-supply may lead to a quicker and better appreciation of high ideals of hygienic purity than more complex methods which require the exercise of special skill in the attainment of good results. Accordingly clear and wholesome natural water

either at or below the surface of the ground is of first consideration, ground-water treated when necessary to improve its quality for general use is of the second rank, and artificially purified surface-water should be acceptable in the absence of facilities for obtaining either of the former grades of water-supply economically or in sufficient quantity.

CHAPTER II.

RIVER-WATER SUPPLY.

Rivers usually constitute a frequent and unfailing source of water-supply and are probably more in demand for this purpose in the central and southern portions than in any other sections of the country. But it is seldom that the river-water can be used without some sort of purification. Purification works add much to the expense of the construction, maintenance, and operation of a system of water-works, and when successfully operated demand no small amount of attention from the management and attendants. In the East, sewage and manufacturing wastes constitute the chief source of river pollution, while in the Middle West, where these sources of pollution are relatively less, there is an exceedingly large amount of sediment carried in suspension by nearly all rivers and streams which needs to be removed. Problems of water purification, while having the same object in view in all sections of the country, receive solution in a manner differing very much in the mechanical details of the purification works. For instance, the filter is usually considered the essential requirement for removing sewage pollution, and the settling-basin or a combination of the settling-basin and filter for clarifying muddy water.

In order to present the practical side of the several methods of water purification, it is necessary to pursue the description considerably in detail, particularly that portion of the description relating to the preliminary treatment of river-water in preparing it for filtration.

Clarification alone is necessary when a naturally muddy water can be made wholesome by removing the sediment which it contains. This is frequently accomplished by the use of the

settling-basin. But when the purity of the water aside from the sediment which it may contain is in question, the water must be subjected to some additional treatment before it can be considered wholesome. The additional treatment usually consists of sand filtration or the use of a germicide either to remove the bacteria infesting the water or to kill them outright by the toxic effect of some applied chemical. The essential object to be accomplished in either case is the removal of objectionable bacteria that may have entered the source of water-supply with any sewage discharging into the source from which the supply is taken.

There is no sharp dividing line between the process of simple clarification and that of bacterial purification of a water, particularly when the natural water contains much finely divided clay in suspension; for then the process which is necessary to secure a complete clarification artificially will often relieve the water of bacterial life to such a degree as to render it safe and wholesome. In showing the difference between a process of simple clarification and one of purification as now practiced, it is necessary to follow out the present methods of treating muddy water somewhat systematically and in detail.

Settling-basins. — The settling-basin as a factor in water clarification has received its most extensive development in cities along the Missouri and Mississippi Rivers. Here the common practice for years has been to depend upon sedimentation for clarification. Accordingly the basin is usually divided into at least four divisions, each division having a capacity for water of about one day's consumption. Two distinct methods of operating the basin are followed: one is termed the fill-and-draw method, the other the continuous-flow method, the water-supply being abstracted after subsidence from each of the four divisions in the former method and continuously from a single division in the latter method.

The fill-and-draw method embraces a period of uninterrupted rest of the water in one division of the basin while the day's supply is being abstracted from another division, while a third division is being filled, and while a fourth division is being cleaned. A serious and practically insurmountable diffi-

culty in this method of operation is the one of drawing upon the division containing the subsided water for any large percentage of the contained water without disturbing and drawing into the effluent pipe a considerable portion of the intensely muddy lower water which has grown dense with deposits from above. The sediment descends very slowly through the lower water before final repose in a compact form at the bottom of the basin, and when in this semi-fluid state it is very unstable and naturally mingles with the less turbid water from above as it eddies into the effluent pipe. Even a more serious objection to the fill-and-draw plan of operation is the one that a river-water does not respond equally on each and every day to the process of natural sedimentation, as the amount and character of the sediment in river-water varies or as the temperature changes. Within a single day the water in a settling-basin operated on this plan may change from one slightly turbid to one decidedly muddy and opaque. Moreover, a coagulant when needed must be introduced into the raw water as taken from the river by the pumps, and when so introduced much of it is absorbed or subsides with the heavy sediment, leaving the fine sediment practically unaffected by the process of coagulation except when the coagulant is used in excess. The objections mentioned are so serious that the settling-basins originally constructed to operate on the fill-and-draw plan are gradually being remodeled to operate on the continuous-flow plan.

The continuous-flow method of sedimentation provides for the passage of the water from one division of the basin to another successively, the muddy water being introduced into the first division of the series and the clarified water being stored and abstracted from the last division of the series. Progressive sedimentation takes place during the interval of transit from division to division.

The simplest method of communication with the series of divisions of the settling-basin is by long depressions or weirs in the division walls, together with suitable by-pass pipes or conduits to facilitate the emptying and cleaning of any one of the divisions without disturbing the operation of the other divisions of the series.

The water upon entering a division of the basin near the bottom displaces an equal amount of clearer water at the surface which passes to the succeeding division over a weir in a division wall. The weir method of communication between the basin divisions insures a transfer of the clearest water, which is always near the surface, and the least interference with the process of sedimentation. This may properly be termed the displacement principle of the continuous-flow method.

The weirs may or may not be constructed the entire length of a division wall of a basin, the object sought being to convey

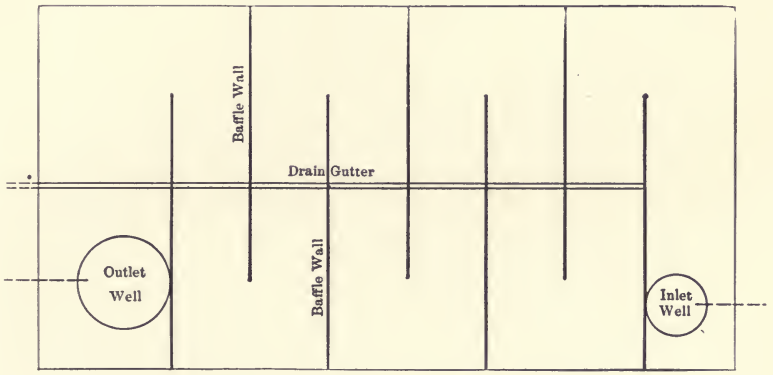


FIG. 15.—Settling-basin with Baffles.

the water from one division of the basin to another in a thin sheet that serves to skim, as it were, the surface-water. The coldest winter weather does not cause any interference of ice at any weir, as the water emerges from under the ice on one side of the division wall, flows over the wall at a temperature slightly above freezing, and disappears beneath the ice on the opposite side of the wall.

A method employed to avoid a tendency to surface currents as the water passes from the weir is to suspend vertically a curtain of planks about two inches from the weir. The curtain deflects the water downward to any desired depth.

A modification of the displacement principle of continuous-flow is to be found in the use of a settling-basin supplied with baffle-walls instead of division walls. These baffle-walls guide the water by a circuitous route through a single compartment

basin upon the theory that the flow, although continuous, is at so low a velocity that sedimentation is progressive during the forward progress of the water. An illustration of this principle is shown

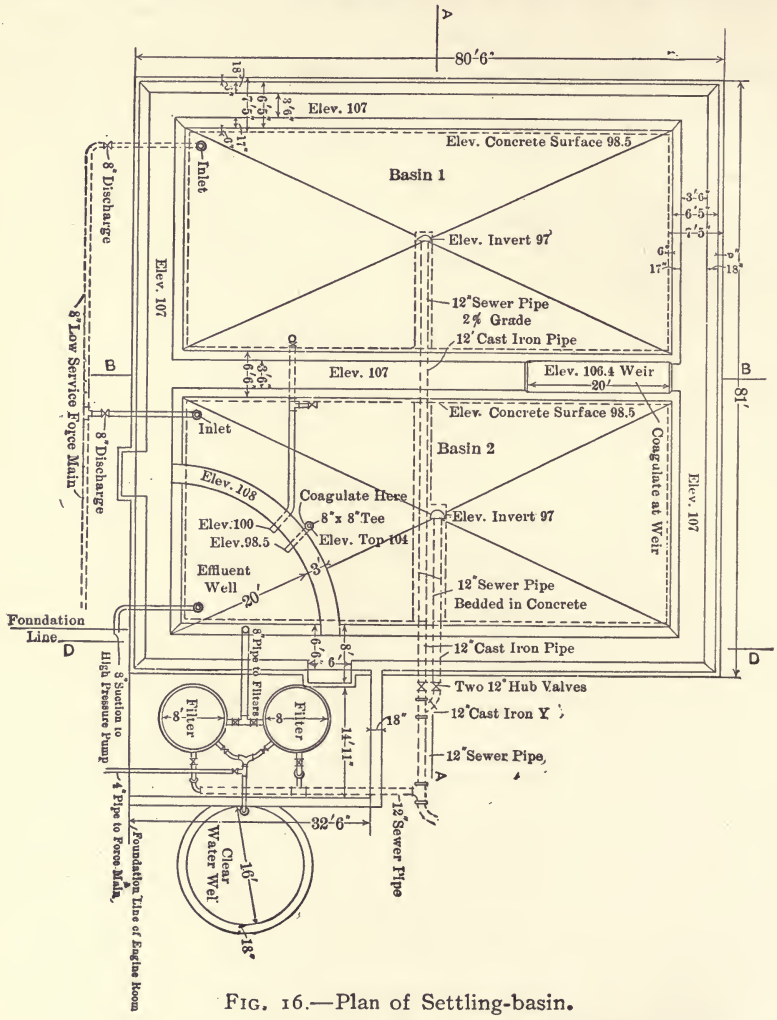


FIG. 16.—Plan of Settling-basin.

on Fig. 15, a sedimentation basin designed for Fort Smith, Arkansas, in 1896 by the author. The water enters one end of the basin, swings around the several baffle-walls, which are built to the same elevation as the top of the basin enclosing walls, and discharges into a receiving-well at the opposite end of the

basin in a thin sheet over the well-rim. Although this structure has never been built, it serves to demonstrate the principle upon which baffles are supposed to work in horizontal flow. A modification of this principle can readily be made to produce a flow in a vertical plane.

Settling-basins having but one or two divisions are some-

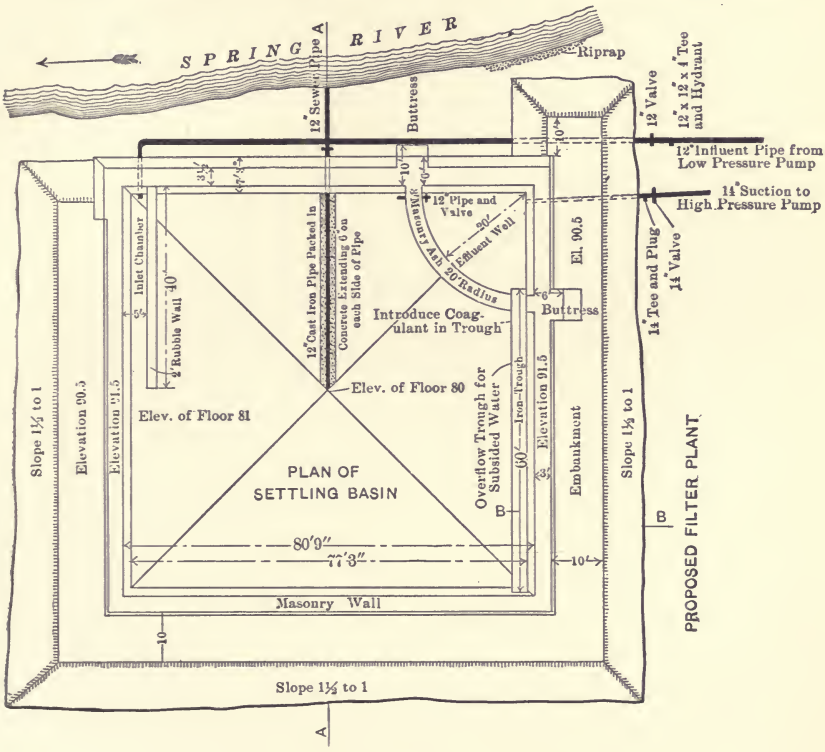


FIG. 17.—Plan of a One-division Settling-basin.

times operated on the displacement method, as shown by Figs. 16, 17, and 18.

Fig. 16 shows a water-purification works consisting of a two-compartment subsiding-basin and a mechanical filter-plant. The raw river-water enters division 1, displacing an equal volume of water which passes into division 2 over a weir, swings through division 2, and finally passes over the arch wall in the corner of division 2 into the effluent well, whence it passes to the mechanical

filters. During the process of cleaning either one of the two divisions of the basin the other division can be operated singly in connection with the effluent well.

A three-division basin (not constructed) operated on the displacement plan is shown in Fig. 19. The principle of opera-

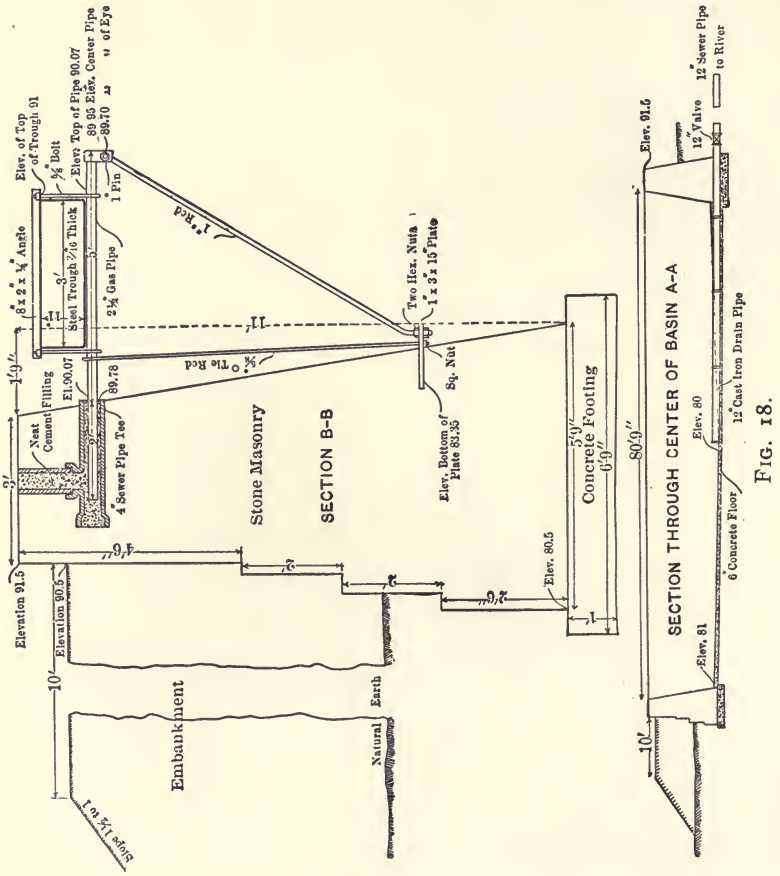


FIG. 18.

tion of this basin is similar to that of the two-division basin just described, except for such modification of operation as is embraced in a three-division layout.

Other designs of settling-basins are to be seen on succeeding figures which accompany descriptions on other pages of this chapter.

Natural Subsidence.—When a muddy water is delivered into a settling-basin operated either by the fill-and-draw method or by that of continuous-flow, sediment naturally settles to the bottom and the water becomes clarified to a degree. This

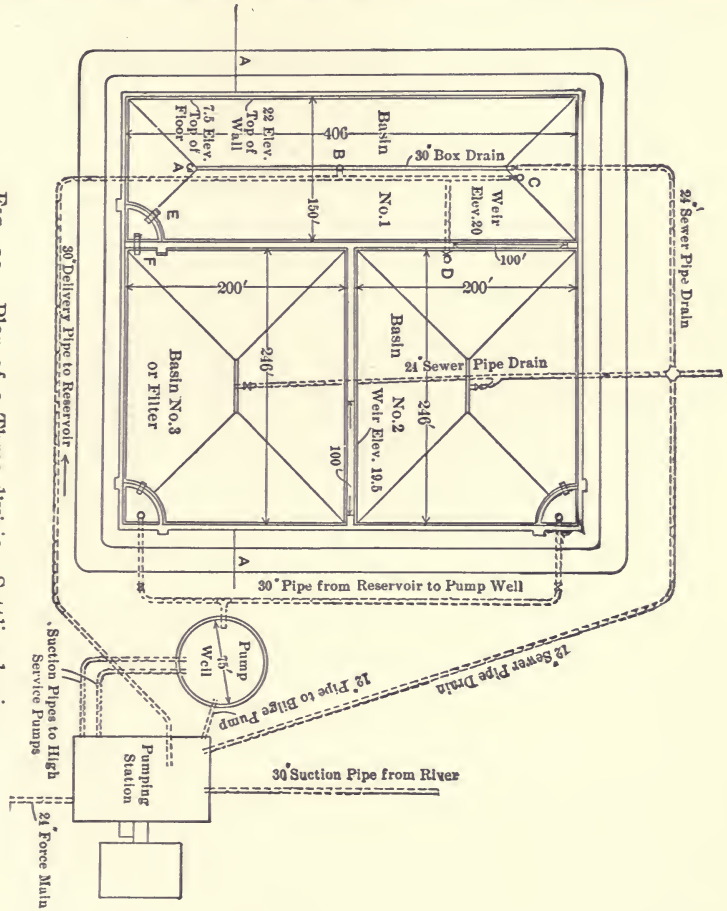


FIG. 19.—Plan of a Three-division Settling-basin.

method of clarification is termed natural or plain sedimentation to distinguish it from the method of sedimentation with the aid of a coagulant.

The degree of clarification which can be attained by natural sedimentation depends upon the character of the sediment in the raw water and to a degree upon the viscosity of the water itself. A sediment composed almost wholly of sand descends

rapidly and a fair degree of clarification may be reached by natural sedimentation. A raw water of this class is characteristic of the Missouri River during the late summer and the fall months, when the sediment carried in suspension is largely sand with a small percentage of clay.

When a considerable amount of clay is carried in the water, there is always a portion of it which is so finely divided and so light that it resists natural subsidence for a long period, even for a longer period than admits of satisfactory clarification in a settling-basin containing four to six times the daily consumption of water. This is the condition of the Missouri River water in the spring of the year and of almost any other detrital rivers of the middle West during either all or a portion of each year, depending upon the character of the soil in the territory drained.

Robert Spurr Weston, resident expert of the Sewerage and Water Board of New Orleans, Louisiana, gives 265 parts per million of suspended matter remaining in the Mississippi River at that city after seventy-two hours' natural subsidence.

In the Cincinnati experiments conducted by George W. Fuller for the purification of the Ohio River water, Mr. Fuller tabulates results of natural subsidence as follows:

TABLE I.—SUSPENDED MATTER IN OHIO RIVER.

(In parts per million.)

Range.	Original.	After 72 hours of plain subsidence.	Percentage removed.
1 to 50	38	12	68
51 to 100	73	19	74
101 to 250	166	48	71
251 to 500	330	93	72
501 to 1000	665	184	72
1000 and over	1235	222	82

In some observations of sedimentation of Missouri River water the results are as shown by Table II.

The observations in this table were made of water passing through the settling-basin of the Kansas City, Missouri, waterworks, illustrated by Fig. 20. Observations after a longer period of natural sedimentation could not be recorded, as the water received a coagulant solution immediately after twenty-four-hours' natural

sedimentation. There were occasions, however, before a systematic method of coagulation was introduced when the water was completely opaque and must have contained several hundred parts per million of sediment after fully four days' exposure to natural subsidence.

TABLE II.

(In parts per million.)

Date.	Suspended matter. River-water.	Suspended matter in River-water after about 24 hours' natural subsidence.	Suspended matter removed by natural subsidence. Per Cent.
January 18, 1900	650	80	87 7
" 20 "	515	100	80.6
" 24 "	490	140	71 4
" 26 "	515	50	90.3
March 10 "	2315	150	93.5
" 12 "	2520	325	87.1
" 14 "	2000	470	76.5
" 19 "	1160	295	74.6
	Average...	202	82.7

At St. Louis, where plain or natural sedimentation has been practiced for a number of years, the results are expressed in the following table.

TABLE III.

Period.	Average sediment in raw water, parts per million.	Average period of subsidence. Hours.	Sediment in subsided water, in parts per million.	Per cent of sediment moved.
Bissell's Point, 1885–86.....	1186	24	104	91
Bissell's Point, 1887–88.....	1186	34½	305	74
Chain of rocks, 1900–1	1152	60	165	86

In the experiments at Louisville with the Ohio River water the average of five experiments was:

	Parts per million.
Suspended matter.....	469
Sediment in subsided water, 24 hours' settlement	277
" " " " 48 " "	155

The per cent of sediment removed is about 41 per cent for twenty-four hours' and 67 per cent for forty-eight hours' natural sedimentation.

The remnant of the sediment still suspended in the water after natural sedimentation is probably of the same general

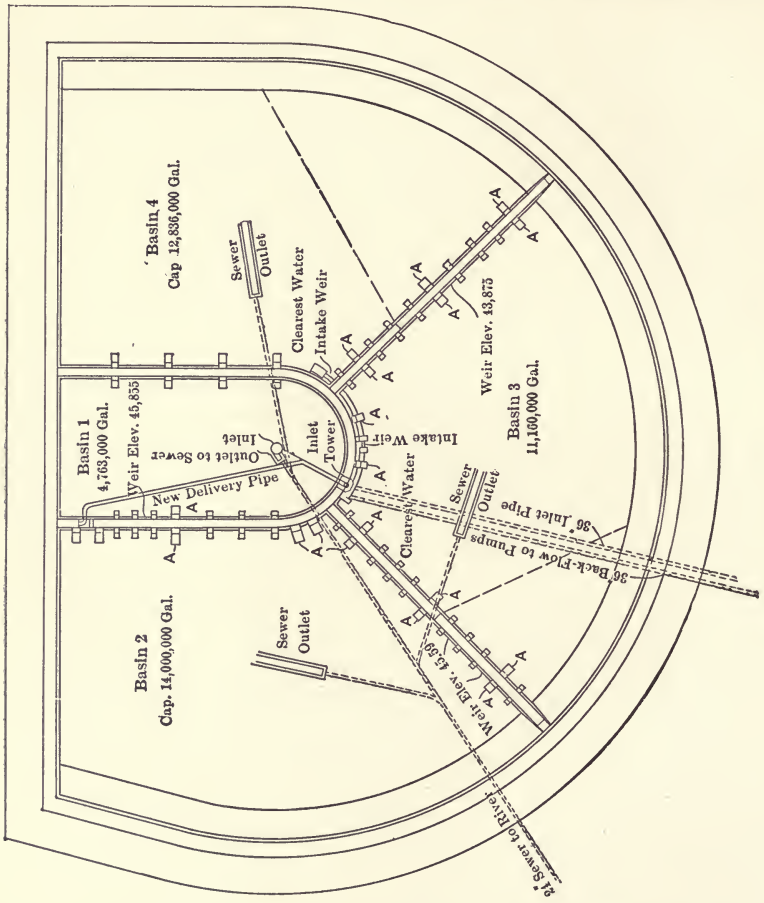


FIG. 20.—Settling-basin at Kansas City.

character in each instance referred to, as the average amount of this remnant does not vary much in any of the river-waters named. A greater variation may be found by comparing the results of natural sedimentation of the several river-waters at seasons of the year when turbidity is most persistent.

Sedimentation is slower in the spring of the year when the

water is chilled because of the greater viscosity of the water at this season of the year. There is no practical way of remedying this difficulty.

It is evident from what precedes that natural sedimentation is not an effective means of thoroughly clarifying muddy river-water, although it is a very effective method of removing the heavy sediment which is simply held in suspension by the complex motion of a flowing river. The remnant of sediment in a naturally subsided water may amount to as much as 1 cubic yard for each 1,000,000 gallons of water or even more.

The result of natural sedimentation for clarifying a public water-supply taken from a detrital river has been found efficient in no city which has thus far tried the process, but notwithstanding the fact that natural sedimentation has its limitations and is a failure in the regard stated it has its proper and necessary place in the art of water clarification.

Circulation.—Phenomena which are apparent in a large settling-basin may be absent in an experimental plant because of the impracticability of duplicating in a small way all the conditions which affect the circulation of water in a large settling-basin. Many of the author's observations recorded in this chapter were made while conducting experiments on a small scale and while supervising the operation of the Quindaro settling-basin of the Kansas City, Missouri, water-works. A plan of the Kansas City basin is shown by Fig. 20. It is about 20 feet deep at the division walls, and as originally designed by G. W. Pearson, division No. 1 of the basin was the receiving-basin of the raw water. The water entering the basin through a rising pipe and a well about 4 feet high, impinged against an inverted conical deflector which caused the water to spread horizontally in all directions. In 1899 the influent pipe was extended across division 1 and was made to deliver the raw water into a conduit in the wall separating divisions 1 and 2, through which it flowed to openings at the foot of the vertical wall of the straight side of the basin and discharged laterally into division 2. The object of this change was to make division 1 a clear-water basin, as it had become too small to work successfully as a raw-water basin when the rate of delivery into it exceeded 6 to 8 million gallons

a day. Modifications were also made of the manner of passing the water from one division of the basin to another at about the same time as the change above noted was made. As originally designed the water passed from one division of the basin to another through conduits and openings constructed in the division walls, the water entering openings near the top on one side of a division wall and discharging into an adjoining basin

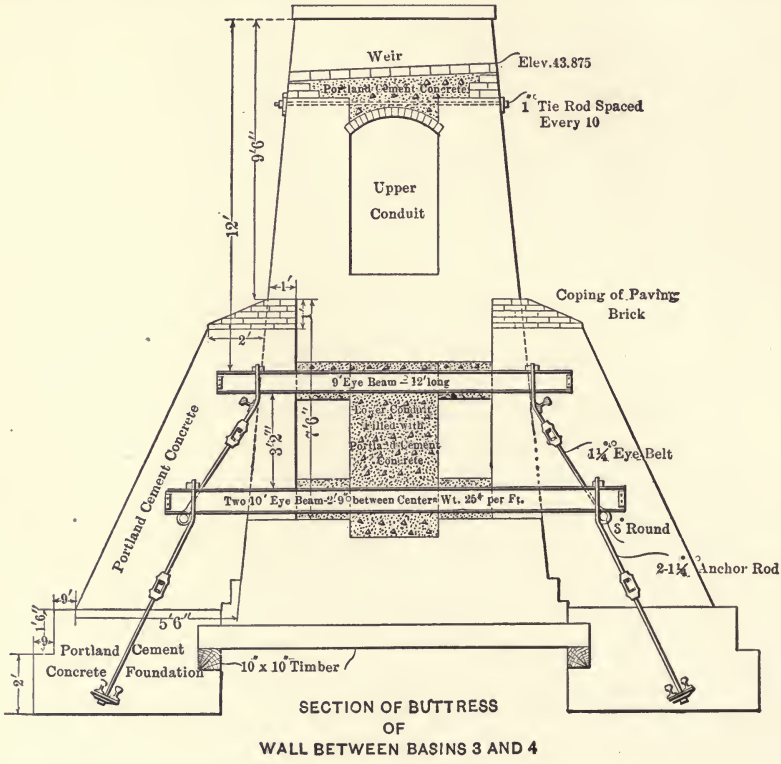


FIG. 21.

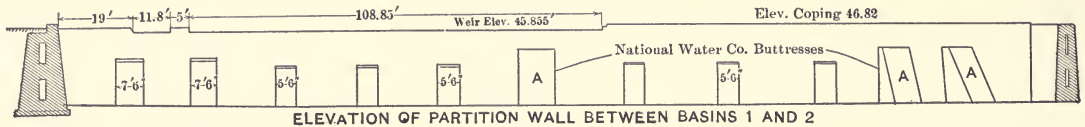
through similar openings at the foot of a division wall. The water circulates on the modified plan by overflowing a portion of a division wall. The water from division 2 flows into division 3 over a weir in the division wall 133 feet long. In a similar manner the water passes into division 4 from division 3 over a weir in the wall between divisions 3 and 4, 185 feet long, shown in section by Fig. 21. From division 4 the water enters division 1

through a conduit in the straight side wall. Provision is also made for delivering the influent water into any one of the three large divisions while some one of these divisions is temporarily out of service. Plates XVI, XVII, and XVIII show an elevation of the weirs in the division walls.

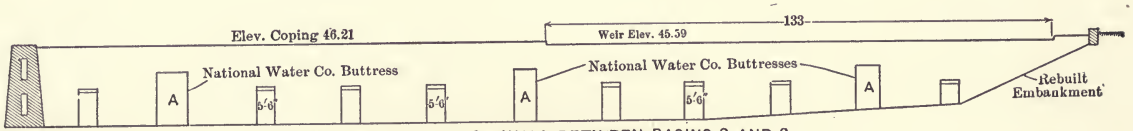
The effluent well remaining as originally designed is a conduit in the bottom of the circular wall between divisions 1 and 3, extending from the inlet tower to the large buttress of the circular wall between divisions 1 and 4. It connects with divisions 1, 3, and 4.

In operating a settling-basin on the displacement plan the influent water should be delivered horizontally near the bottom of the basin in order to avoid as much as possible a tendency to upward vertical motion and to favor a dispersion of the entering water, conforming to a temperature contour.

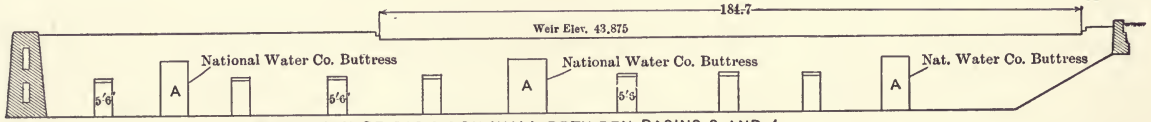
The entering water when thus introduced distributes laterally over the basin without rising to the surface. This fact has been observed upon numerous occasions by comparing samples of water from various depths and from various portions of the division receiving the raw river-water. For a considerable depth the turbidity is about uniform, then slightly increases, and finally there is a decidedly abrupt change from turbidity to intense muddiness. The variations of the depth of the mud plane appear uniformly distributed throughout the entire division receiving the raw water, except when the entering water is abruptly deflected upward near the delivery openings, where the heavy sediment is chiefly deposited. Below this mud plane there is a thick fluid mud which apparently increases in density until the solidified deposit on the bottom is reached. As the deposit accumulates, the level of the mud plane rises and eventually mud streaks the overflow into the succeeding basin, indicating the approach of cleaning time. However, the working life of the raw-water division may be somewhat prolonged by discharging a portion of the liquid mud through the drain. The working of a settling-basin on the displacement plan continues satisfactorily until the basin becomes foul or overtaxed, or until vertical currents are induced by a sudden and injudicious increase of the rate of delivery of river-water into the basin, or until a rapid



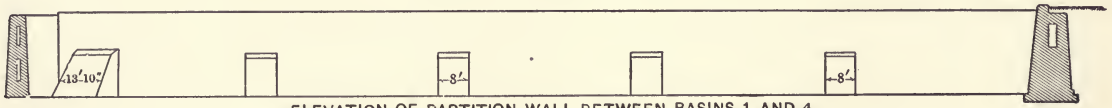
ELEVATION OF PARTITION WALL BETWEEN BASINS 1 AND 2



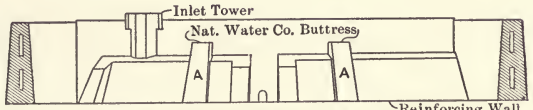
ELEVATION OF PARTITION WALL BETWEEN BASINS 2 AND 3



ELEVATION OF PARTITION WALL BETWEEN BASINS 3 AND 4

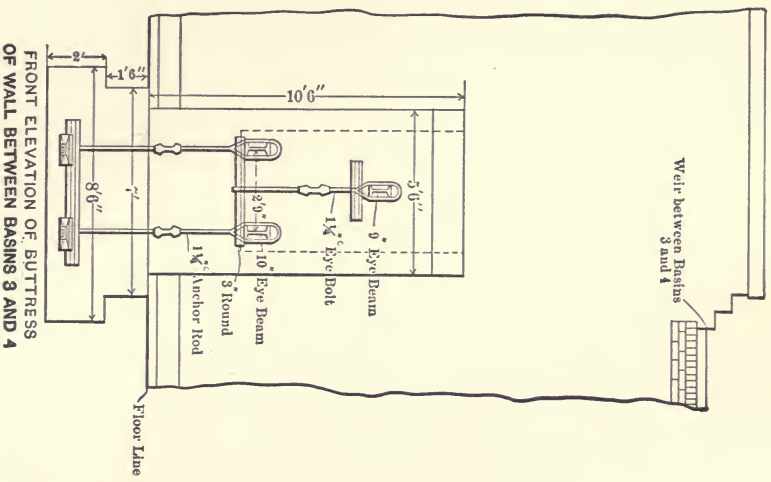


ELEVATION OF PARTITION WALL BETWEEN BASINS 1 AND 4



WALL BETWEEN BASINS 1 AND 3

PLATE XVI.


Weir between Basins 3 and 4
10′6″
5′6″
1¼″ Eye Bolt
9″ Eye Beam
2′6″
10″ Eye Beam
3″ Round
1¼″ Anchor Rod
Floor Line
2′
1′6″
8′6″
FRONT ELEVATION OF BUTTRESS OF WALL BETWEEN BASINS 3 AND 4

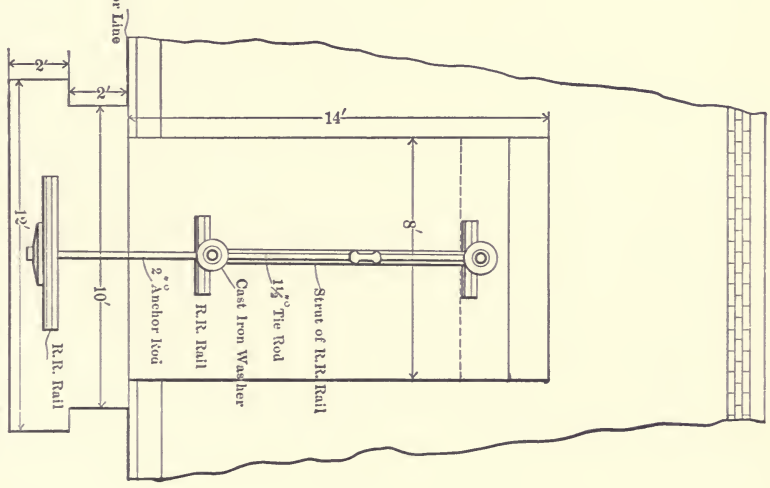
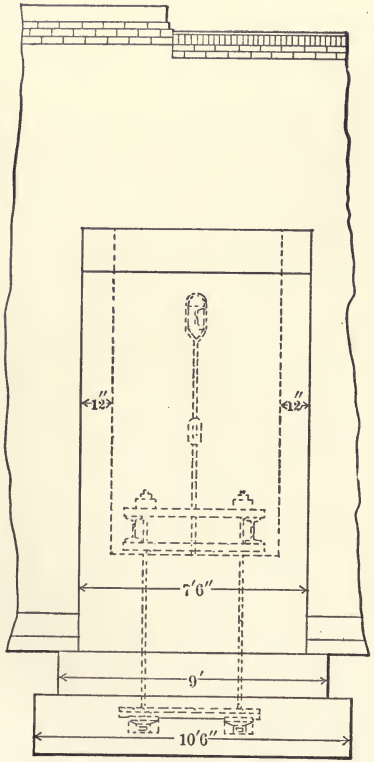
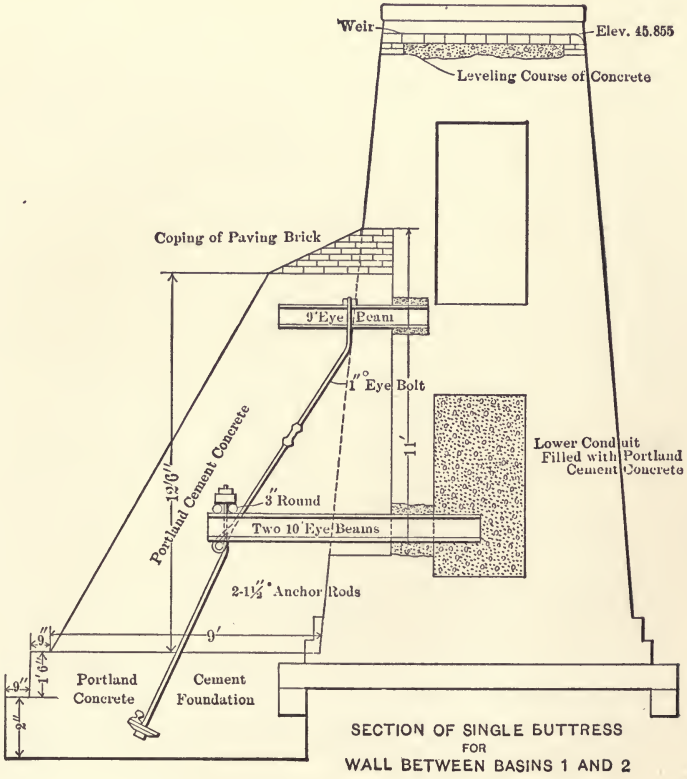

Floor Line
2′
2′
14′
8′
10′
1½″ Tie Rod
Strut of R.R. Rail
Cast Iron Washer
R.R. Rail
2″ Anchor Rod
R.R. Rail
FRONT ELEVATION OF LARGE BUTTRESS

PLATE XVII.



FRONT ELEVATION OF BUTTRESS



SECTION OF SINGLE BUTTRESS FOR WALL BETWEEN BASINS 1 AND 2

PLATE XVIII.

building up of the heavy sediment near the influent openings deflects the entering water abruptly upward.

The author's attention was early directed to the theory of displacement as producing the continuous flow of water from one division of a settling-basin to another by observations of the tendency of the water entering a settling-basin to stratify according to a temperature contour. Turbidity observations made of the raw-water division of the Kansas City settling-basin showed that there existed at the time of one series of observations but slight difference in the degree of turbidity for the first 13 feet in depth and almost a uniform degree of turbidity for the first 5 feet, even over the influent apertures. The transition from a state of turbidity to one of dense muddiness was sudden and pronounced below the 13-foot level. Other observations of the same kind at different times demonstrated similar conditions with the mud plane at a quite uniform level, though of a varying depth from week to week.

It was also observed when a division of the basin was empty for cleaning that the solidified fine mud was quite uniformly distributed over the bottom, sometimes exhibiting a stratification of different shades, while the heavy sediment was found to form a comparatively high embankment in the vicinity of the inlet opening.

When pumping machinery is employed to deliver the raw water into a settling-basin a thoughtless attendant will occasionally speed the pumps unduly, with the effect of abruptly changing the rate of delivery and of unbalancing the adjustment of lateral circulation. The increased energy thus communicated to the water often induces convection currents which give an upward circulation of water above the mud plane established by the previous régime of pumping and circulation, with a corresponding infusion of intensely muddy water into the partially clarified water of the upper depths of the basin. The convection currents readily climb the faces of the enclosing walls and slopes of a basin and diffuse laterally therefrom.

The tendency of water to stratify is very marked during the summer and autumn seasons when the temperature of the air is either comparatively steady or subject to gradual change.

Then the upper layers of the water in the basin reach a temperature above that of the river-water, and the entering river-water, responding to the law of gravity, naturally seeks the lower depths and a lateral distribution in the basin corresponding to some temperature contour. Doubtless this stratification of the water accounts for the existence of the mud plane referred to on preceding pages.

At other seasons, particularly the early spring and the early winter months, certain natural causes serve to break in upon the normal working condition of the settling-basin, producing pronounced convection currents which are locally termed "boiling." The appearance of this boiling is not due to any sudden alteration of the régime of operating the basin, as all divisions of the basin may be similarly affected, though perhaps not in the same hour or day. Temperature observations made of the water in the Kansas City settling-basin revealed temperature conditions at different depths of the basin indicated by Table IV.

TABLE IV.

Date.	Depth below water-surface in subsiding-basin in feet.									Near bottom.	River.
	0	1′	2′	3′	4′	5′	6′	10′	11′		
	Temperature in degrees Fahrenheit.										
June 2, 1898.....	82	82	82	82	81	80½	80½	80½	80		80
July 25, 1898.....	84							82			
Aug. 10, 1898.....	87					84		83			
Sept. 8, 1898.....	76					72		71			72
Apr. 14, 1899.....	54								52		
Jan. 20, 1900.....	36									36	
Jan. 26, 1900.....	36									38	

In every instance noted in the table except that of January 20th, the temperature observations were made with the basin in a stable working condition and unaffected to any material degree by convection currents. On January 15, 1900, the divisions all exhibited pronounced convection currents, particularly division 3. In division 4 much hydrate of alumina boiled up to the surface of the water. On January 16th the convection currents had nearly abated. At that time the surface tempera-

ture was 35 degrees, and the temperature was 36 degrees at a depth of 15 feet. On this date the hydrate of alumina in division 4 had largely disappeared. On January 20th the boiling action was resumed in division 3 and was clearly visible along the wall separating divisions 3 and 4, along the west embankment, and in various other parts of the division. A thin skin of ice which had formed over the basin during the preceding night emphasized the appearance of the convection currents as they were revealed through the ice. The ascending mud and gray hydrate of alumina seemed to rise with a rotary motion and after contact with the ice rolled away radially in a manner resembling the rotating smoke and steam blown from a locomotive stack as it gradually rises, expands, and disperses through the atmosphere.

Temperature observations made on that date at the surface and 15 feet below the surface revealed a uniform temperature of 36 degrees.

Doubtless the same tendency towards stratification exists in the water as in the atmosphere. When a break of the stratification ensues, the vertical currents are similar in both fluids, though of course less violent in water because of its greater weight and inertia.

Numerous temperature observations of the kind indicated in Table IV were made at various times, but all seem to indicate that a fall of temperature of about two degrees or more between the surface-water and that about 10 feet below is necessary to preserve a stable working condition of the water in a basin operated upon the displacement method. Convection currents develop frequently in the spring of the year as the ice in the river breaks up and moves out, but before the ice in the settling-basin has entirely melted, as well as in the late fall or early winter when new ice is forming or the surface of the water in the basin becomes chilled, and in this regard the movements of the water resemble the seasonal upsetting of the water in a storage reservoir or in natural lakes. When vertical circulation is in progress the upheaval of sediment from the deeper portions of the water of a settling-basin temporarily emphasizes the difficulties of water clarification by sedimentation. Even a deep basin, although

advantageous in several respects, may be no protection from temperature convection currents. The use of deep settling-basins has been discouraged because of the expense of constructing high masonry walls. This objection can now be met in a measure at least by the use of reinforced concrete.

Surface currents, usually pronounced when the water passes off a weir in a division wall at the water-level of an adjoining basin, may be prevented by a curtain of planks set several inches from the face of the division wall and projecting several feet beneath the surface of the water, thereby serving to improve circulation and to assist clarification.

Baffles have been introduced in small subsiding basins to direct the circulation of water, but as simplicity of design and operation is desirable in sedimentation basins intended for operation on a large scale baffles complicate the operation and add considerably to the expense of construction and maintenance.

The swinging motion of the water as a body is still another form of circulation which seems to be entirely independent of those herein described. The motion is so slow that evidence of it is to be found rather in the relative clearness of the water in different portions of any division of a basin than in any well-defined visible movement. In the Kansas City basin the clearest water to be found in either division 3 or 4 is in the corner of the divisions marked on Fig. 20 "clearest water," opposite the outside or boundary walls of the basin. In this case the water after falling from the weir seemed inclined to swing toward the boundary walls and to clarify as it receded from the weir and finally to reach the highest degree of clearness when it had reached the location of the zone of the clearest water above alluded to. The line of separation of the clearest water in the basin from that discharged from the weir is generally very decided and distinct. What influence the peculiar form of the basin referred to may have in producing this swinging motion has not been investigated, but there is every reason to believe similar conditions would exist in a rectangular basin were the weirs of no greater fraction of the length of the division walls than they are in this basin. It is quite probable, however, were the weirs to extend the whole length of a division wall the swinging

movement of the water as a body would become less apparent.

It cannot be assumed that the basin is effective only for a width equal to the length of a weir in a division wall and that the water is dead in the corners of a basin beyond the ends of a weir, for it has been observed that a change of turbidity in the water falling from a weir is followed by corresponding but progressive change in all parts of the division receiving the water, even in all the corners, showing that all portions of the basin are effective. The corner of "clearest water" is the last, of course, to fall under the influence of any change, but none the less sure of showing the degree of change in the division as a whole. Insistence upon a symmetrical or geometrical figure of a settling-basin seems unnecessary, and in fact is frequently impossible owing to topographical or land boundary considerations, nevertheless capacity to the fullest extent is serviceable or can be made serviceable by proper design and manipulation of the operating details.

Coagulation.—We have already pointed out the inefficiency of natural sedimentation to properly clarify water of a detrital river. The condition of the subsided water when the beneficial effect of natural subsidence practically ceases is variable, depending upon the amount of silt or finely divided clay the river-water may carry in suspension. So great is this variation that exposure of a raw river-water to a stated number of hours of natural sedimentation is found to show a turbidity variation of considerable range. For instance, twenty-four hours' natural sedimentation will afford a much clearer water at some seasons of the year than at other seasons and during some periods of the same season than during other periods. So deficient has natural sedimentation been found that the aid of a coagulant is necessary to produce even a fair degree of clarification on many occasions. The amount of coagulant used varies with the degree of turbidity of the naturally subsided water. Thus far the sulphate of alumina has been used most extensively to facilitate clarification.

Clarification of the water with the aid of sulphate of alumina is both a chemical and mechanical process, and as such it is now so well understood as to scarcely require explanation in

this chapter were it not that the various allusions herein made seem to require some general explanation.

The use of a coagulant is for the purpose of gathering together into small masses the very fine particles of silt and clay which of themselves are so light and fine as to resist natural subsidence and to thereby hasten their fall through the water.

The coagulant for successful use must be harmless in its effect upon the water treated, leaving its wholesomeness unimpaired, and also cheap enough in the market to be available to any community which by force of circumstances may have occasion to use it in the clarification of the water-supply. The sulphate of alumina, when judiciously used, has been found to answer the requirements successfully. Its properties and its effect upon turbid waters have long been known. As manufactured, it is a salt of aluminum composed of about 16 to 18 per cent of alumina, about 36 to 37 per cent of sulphuric acid, and about 44 to 46 per cent of water, together with a small percentage of impurities like oxide of iron, in a chemical combination similar to a combination of sulphuric acid with lime or magnesia. In a salt properly made, there should be no free sulphuric acid. It is highly soluble in water and therefore readily made into a solution of any degree of strength up to one of complete saturation of the water.

The chemical affinity of the constituents of this salt is comparatively weak and is readily broken in the presence of other solutions of mineral salts, such as the bicarbonate of lime and magnesia, and made to assume new chemical combinations of a more stable character. In fact its use depends upon the presence in the water to be treated of a mineral salt in solution capable of new chemical combinations with the applied solution. Many natural waters contain abundance of lime and magnesia, the constituents of limestone, which are held in solution by carbonic acid, and with these salts the dissolved sulphate of alumina readily enters into combination, forming sulphate of lime or magnesia, free carbonic acid, and hydrate of alumina. The sulphate of lime or magnesia and the carbonic acid remain dissolved in the water, while the hydrate of alumina, being insoluble in water, remains a floating though finely divided solid and

thus completes the essential chemical part of the process of coagulation.

The presence of the carbonates of lime is characteristic of natural waters in limestone territories and is familiarly termed "temporary hardness" or alkalinity, the qualifying term "temporary" being used to distinguish the portion of the lime precipitated by boiling, the boiling having the effect of releasing the carbonic acid as a gas. Natural waters which are soft or possess little or no alkalinity must first be treated with lime water, a solution of carbonate of soda, or some mineral base of this character before the sulphate of alumina solution is introduced.

The sulphate of lime and magnesia is a usual constituent of natural waters and represents the permanent hardness of water. In contrast with the temporary hardness, the permanent hardness is not affected by boiling.

Carbonic acid as found in natural waters is acceptable, it is desirable in a drinking-water and is present in natural waters in varying amounts.

It should be observed that the use of the sulphate of alumina introduces no new chemical compounds in solution in the water treated; practically its only effect is to transfer a portion of the mineral water in solution from a state of temporary hardness to one of permanent hardness, but usually the amount of mineral thus affected is small in comparison with the total amount of hardness naturally present in the water treated. Of course no undecomposed sulphate of alumina should remain in the water after treatment and does not remain when the coagulant is intelligently used.

The hydrate of alumina formed by the chemical combination just described is a sticky insoluble mineral in a finely divided state when freshly formed, but gradually the small particles unite as they circulate through the water, collect the fine sediment, and finally subside in small masses. This constitutes the mechanical part of the process of coagulation.

A common practice is to introduce the coagulant solution into the raw water as it is being delivered to the settling-basin. In cases where the water is delivered to the basin by pumping,

a small by-pass pipe is connected with the main delivery-pipe and with a pressure-tank. Dry sulphate of alumina is put into the tank and the top bolted on. Water from the delivery-pipe is then turned into one end of the tank, dissolving the sulphate of alumina as it circulates through the tank; the solution passing out at the opposite end of the tank is returned to the delivery-pipe. This method of coagulation is exceedingly faulty and crude, as it gives a very irregular distribution of the coagulant. Introduced in this manner the coagulant has apparently little or no effect in clarifying very muddy water because the hydrate of alumina is either absorbed by the large amount of organic matter usually present in muddy water or becomes entangled with the heavy sediment as it subsides naturally and goes to the bottom of the basin without effect on the fine sediment.

Several years ago the author made a few simple analyses to determine the loss of coagulant when introduced into the raw Missouri River water, and to determine a ratio of the alkalinity of the water consumed in the process of coagulation to the amount of coagulant used in the operation. The method employed in these experiments was to collect a sample of the raw river-water as it passed through the basin delivery-pumps and to treat immediately one portion of this sample with a definite amount of coagulant solution of known strength, and another equal portion of the same sample after removing the sediment by filtration with an equal amount of the same coagulant solution. After treatment and subsidence, an aliquot part of each sample was siphoned off and its alkalinity determined and compared. The difference of the residual alkalinity of the two samples respectively was taken to represent the equivalent of the coagulant absorbing capacity of the sediment and was reduced to grains per gallon by dividing it by the ratio of the alkalinity consumed in the treatment of the filtered sample to the known amount of coagulant in grains per gallon used in the treatment. The results are compiled in Table V.

In the experiments indicated by the results collated in Table V an effort was made to keep a little within the range of the total coagulant decomposing capacity of the river-water. A proper

TABLE V.

Date. 1900	Description of sample.	Suspended matter in river-water, parts per million.	Sulphate of alumina, grains per gallon.	Alkalinity. Before treatment.	Alkalinity. After treatment.	Ratio of alkalinity consumed to coagulant used, grains per gallon.	Coagulant absorbed by mud, grains per gallon.
Feb. 5		30	20·5	230	53·5	8.6	
" 9		30	26.1	252·5	25	8.7	
" 21		30	23.2	255	50	9.0	
March 2		30	23.2	230	27·5	8.7	
" 5		30	23.2	230	30	8.6	
" 7	Muddy	740					
" 8	Muddy		17·4	175	43·5		
	Filtered		17·4		35·0	8.0	
" 9	Muddy	1220	15·5	145	52·5		
	Filtered		15·5		25·0	8.4	3·3
" 10	Muddy	2315	11.6	120	37·5		
	Filtered		11.6		25·0	7.8	1.6
" 12	Muddy	2520	11.6	125	65·0		
	Filtered		11.6		35·0	7.8	3.8
" 13	Muddy		11.6	120	55 0		
	Filtered		11.6		35·0	7·3	2.7
" 14	Muddy	2000	11.6	115	45·0		
	Filtered		11.6		25	7·7	2.6
" 15	Muddy		11.6	112.5	55		
	Filtered		11.6		25	7·5	4.0
" 17	Muddy	1500	11.6	116.5	55		
	Filtered		11.6		30	7·5	3·3
" 19	Muddy	1160	11.6	122.5	50		
	Filtered		11.6		37·5	7·3	1·7
" 20	Muddy		11.6	127·5	55		
	Filtered		11.6		22·5	9·4	
" 21	Muddy	2520	11.6	136.5	67·5		
			11.6				
" 22	Muddy		11.6	150	75		
	Filtered		11.6		52·5	8.4	2·7
" 23	Muddy		14·5	150	52·5		
	Filtered		14·5		40	7 6	1.6
" 24	Muddy	510	14·5	150	47·5		
	Filtered		14·5		40	7.6	1.0
" 29	Muddy	1750	14·5	160	90		
	Filtered		14·5		50	7.6	5·3
April 28	Muddy	3420	14·5	125	75		
	Filtered		14·5		22·5	7·1	7·4
" 30	Muddy		14·5	122·5	50		
	Filtered		14·5		17·5	7·2	4·5
					Average	7·9	

test showed that no undecomposed coagulant remained in the water after each treatment. What difference there might have been in the relation of the residual alkalinities in each set of experiments had a small amount of coagulant been used was

not determined. The object in view was simply to confirm previous observations, made during the actual working of a large settling-basin, of the ineffectiveness of introducing the coagulant directly into the raw river-water heavily burdened with sediment. The ratio of the alkalinity consumed by the coagulant to the number of grains of coagulant used in the treatment of the water agrees fairly well in all instances except that for March 21st, which ratio is omitted in determining the average ratio of 7.9. It is perhaps practically correct to use a ratio of 8 to conveniently determine the capacity, of the Missouri River water at least, to decompose sulphate of alumina, and in such computations as follow this ratio has been used. The ratio is only approximated, however, when the amount of coagulant used in treating the water approximates the total coagulant decomposing capacity of the river-water, as is illustrated by the following table. The water in the subsiding basin referred to above on January 20, 1900, having an alkalinity of 210 parts per million, when treated with different amounts of sulphate of alumina showed a variable ratio of alkalinity consumed to the amount of coagulant used in the treatment. The results were verified by analyses on several succeeding days.

TABLE VI.

Amount of sulphate of alumina in grains per gallon.	Alkalinity after treatment, parts per million.	Ratio of alkalinity consumed to coagulant used.
5.8	146	11.0
8.7	122	10.0
11.6	102	9.3
14.5	80	9.0
17.4	63	8.5
20.3	36	8.1
23.2	26	7.9
26.1	10	7.7

The results of the experiments recorded in Table V not only show the extreme wastefulness of the practice of introducing the sulphate of alumina into a raw water heavily charged with sediment, but also offer another reason for the unsatisfactory results to be derived from the fill-and-draw method of operating a settling-basin which admits only of the coagulation of

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the raw water as it is delivered into the basin. An attempt to use an amount of coagulant in the clarification of a very muddy water which provides for the losses above pointed out and in addition reaches and coagulates the fine sediment which gives persistent turbidity to a water after natural sedimentation entails a prohibitive expense in clarifying water on a large scale.

The most effective time to introduce the coagulant is after the water has passed through a period of natural sedimentation. The length of this period varies with the character rather than the amount of the sediment contained in the raw water and with the season of the year and should scarcely exceed twenty-four hours in length.

A coagulant introduced into the Missouri River water after about 83 per cent of the sediment had been disposed of by about twenty-four hours' natural sedimentation has proven very effective. The amount of coagulant required for clarification is found to vary with the turbidity after the twenty-four-hour period of natural sedimentation, but not necessarily with the amount of sediment in the raw water.

In this connection and as a matter of illustration it may be of interest to tabulate a few analyses indicative of the character of Missouri River water and of the amount of sulphate of alumina that is required for clarification.

The alkalinity recorded in the second column of Table VII is a measure of the coagulant decomposing capacity of the river-water expressed in parts per million, and in column 4 the same capacity is expressed in grains per million and is found by dividing the alkalinity by 8, the ratio derived from Table VI. The table embraces the period of the least as well as the greatest alkalinity of the river-water for the interval of a year. The large surplus of alkalinity beyond that required for chemical union with the sulphate of alumina with which the water was treated affords conclusive proof that no undecomposed sulphate of alumina could have remained in the clarified Missouri River water of that year.

An absence of turbidity observations in the table is noticeable and the observations may accordingly be considered incomplete when viewed from a scientific standpoint. A few tur-

TABLE VII.

Date.	In parts per million.		In grains per gallon.		Alkalinity, city water parts per million.
	Alkalinity of river-water.	Suspended matter, river-water.	Coagulant decomposing capacity, river-water.	Coagulant used in subsiding-basin.	
Apr. 23, 1899..		5350			
24 " ..		3584			
26 " ..	126	3491	15.8	2.18	120.75
27 " ..	128.6	2787	16.1	1.9	112.9
28 " ..	115.5		14.4	1.9	
May 1 " ..		3013			
2 " ..	120.7	5279			
3 " ..	131.2		16.4	1.7	
5 " ..	123.4	4221	15.4	1.7	
17 " ..	126		15.8	1.55	
19 " ..	126	2446	15.8	1.82	
22 " ..	113.9	5230	14.2	2.76	
" " ..	110.2		13.8	2.40	
25 " ..		2530		2.83	
June 10 " ..	128.6	8064	16.1	0.95	110.2
19 " ..	120.75	5756	15.1	1.62	
July 11 " ..	120.7	3633	15.1	0.95	110.2
28 " ..	115.5	3418	14.4	0.69	
Aug. 7 " ..	107.6		13.4	1.19	103.7
8 " ..		2420		1.00	
10 " ..	105	2340	13.1	0.75	103.7
13 " ..	105		13.1	1.30	
22 " ..	120.7	1735	15.1	0.96	
Oct. 13 " ..	159.3		19.9	0.37	
21 " ..	155		19.4	0.53	150
Nov. 1 " ..	157.5		19.7	0.42	155
6 " ..		660		0.44	
8 " ..	160		20	0.41	157.5
Dec. 18 " ..				0.23	170
21 " ..	210	50	26	0.22	177.5
30 " ..	240		30	0.0	225
Jan. 3, 1900..				0.0	265
5 " ..	275		34.4	0.0	
8 " ..	275		34.4	0.0	
9 " ..	260	120	32.5	0.23	
11 " ..	245		30.6	0.42	
12 " ..	265	370	33.1	0.45	
13 " ..				0.55	252
15 " ..	210	830	26.0	0.75	235
16 " ..	215		26.9	0.76	227
18 " ..	215	650	26.9	0.45	230
20 " ..	210	100	26.0	0.43	227
22 " ..	195		24.4	0.50	210
24 " ..	197	490	24.6	0.50	210
26 " ..	192	515	24 0	0.44	205
27 " ..	190		23.8	0.42	192
29 " ..	205		25.6	0.45	198
31 " ..	225		28.0	0.46	195

TABLE VII (*continued*).

Date.	In parts per million.		In grains per gallon.		Alkalinity, city water, parts per million.
	Alkalinity of river-water.	Suspended mater, river-water.	Coagulant decomposing capacity, river-water.	Coagulant used in subsiding-basin.	
Feb. 1, 1900..	222.5		27.8	0.39	200
3 " ..	225		28.1	0.21	217.5
5 " ..	230		28.8	0.0	220
6 " ..	242.5	30	30.3	0.0	225
9 " ..	252.5	30	31.6		240
12 " ..	255	30	31.9		250
14 " ..	255	30	31.9		252.5
19 " ..	260	30	32.5		255
21 " ..	255	30	31.9	0.0	257.5
Mar. 2 " ..	230	30	28.8	0.22	240
5 " ..	225	30	28.1	0.24	232.5
7 " ..	172.5	740	21.6	0.66	230
8 " ..	175		21.9	0.74	210
9 " ..	145	1220	18.1	0.77	195
10 " ..	120	2315	15.0	1.28	175
12 " ..	125	2520.	15.6	2.34	155
13 " ..	120		15.0	3.61	135
14 " ..	115	2000	14.4	2.84	117.5
15 " ..	112.5		14.1	2.43	102.5
16 " ..	120		15.0	1.89	97.5
17 " ..	116.5	1500	14.4	1.72	100
19 " ..	122.5	1160	15.3	2.83	105
20 " ..	127.5		15.9	2.64	105
21 " ..	136.5	2520	17.1	1.61	107.5
22 " ..	150		18.8	1.66	107.5
23 " ..	150		18.8	1.52	125
24 " ..	152	510	19.0	1.43	122.5
26 " ..	150		18.8	1.41	127.5
29 " ..	160	1750	20.0	2.24	142.5

bidity observations were made in the laboratory, but were discontinued for the reason that turbidity observations of the water in the basin itself offered a better guide for the attendant in constant charge in proportioning the strength of a day's coagulant solution. An indication of a progressive increase of turbidity in division 3 of the settling-basin was followed by an increase of the amount of coagulant used until the increase of turbidity was checked, then the strength of the coagulant solution was reduced accordingly as the decrease of turbidity became apparent in division 3.

The variation of turbidity in either division 3 or 4 of the basin was very apparent to a close observer of the daily work-

ing of the basin chiefly in a triangular patch of water extending from the inner end of the weir in either division wall toward the outer wall of the basin, as indicated on Fig. 20. There was usually a rather sharp line of division between the recently coagulated water and that of the water which had undergone partial subsidence after coagulation. A tendency of this line of division to move away from the weir invariably indicated a gradual increase of turbidity, and if this tendency remained unchecked by the use of too small an amount of coagulant the line of division would gradually encompass a larger and larger patch of the water and finally fade almost entirely in a cloud of general turbidity. Thus an attendant making numerous daily observations of the appearance of the water in the several divisions and having presented to his eye the relative position of the line of division between the recently coagulated water and that partially subsided as the line advances with the general swing of the water as a body or retreats toward the weir, and having fixed objects beneath the surface of the water in the clear-water basin by which he could instinctively compare turbidities of succeeding observations, could not fail to detect slight variations of turbidity and to act accordingly day by day in proportioning the strength of the coagulant solution. While this fact and these circumstances account for the absence of a turbidity record in Table VII, they are not to be taken altogether as a justification for a neglect to make frequent laboratory measurements of turbidity not only as a matter of record but also as a help in the economic use of a coagulant. But however this may be it must be acknowledged that the laboratory observations cannot supplant those observations which the attendant can and must make of the working condition of a settling-basin from time to time as he moves along the division walls and sees the operation from different points of view. Instinctively his judgment as to the variations in the management of the basin is largely governed rather by his general observations of the working of the basin itself than by laboratory measurements of turbidity. The laboratory measurements enable him to record the facts or circumstances which influence his judgment.

Observations of alkalinity are the only sure guide to the

safe use of a coagulant when the raw water is comparatively soft or of low alkalinity contents or during periods of floods, particularly flashy floods of rivers having naturally hard water, for then the alkalinity may run abnormally low at a time when there arises the greatest necessity for the use of the coagulant. The coagulant decomposing capacity of the raw water should always be known to be sufficient for the complete decomposition of the amount of coagulant required for clarification, and when at critical periods the alkalinity runs too low to completely neutralize the coagulant, then the requisite amount of alkalinity must be supplied artificially by the introduction of lime-water or a solution of carbonate of soda or other alkaline base.

TABLE VIII.—CONSUMPTION OF SULPHATE OF ALUMINA IN THE KANSAS CITY SUBSIDING-BASIN IN GRAINS PER GALLON.

Day of Month.	1899.									1900.			
	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.
1	1.36	1.78	0.69	0.93	1.00	0.48	0.46	0.42	0.53	0.00	0.39	0.21	1.67
2	1.09	1.72	0.72	1.06	0.69	0.35	0.42	0.42	0.54	0.00	0.37	0.22	1.42
3	1.14	1.70	0.65	0.99	0.75	0.37	0.42	0.44	0.30	0.00	0.21	0.23	1.24
4	1.14	1.56	0.93	1.54	0.69	0.44	0.42	0.44	0.79	0.00	0.00	0.26	
5	1.14	1.70	0.83	1.13	1.04	0.36	0.44	0.42	0.98	0.00	0.00	0.24	1.67
6	1.03	1.78	0.93	1.00	1.19	0.37	0.44	0.44	0.87	0.00	0.00	0.25	1.64
7	1.43	1.71	0.86	1.02	1.19	0.35	0.42	0.42	0.60	0.00	0.00	0.66	1.37
8	1.36	1.71	0.89	1.00	1.00	0.38	0.56	0.41	0.43	0.00	0.00	0.74	1.0
9	1.63	1.83	0.96	1.00	0.73	0.42	0.48	0.20	0.55	0.23	0.00	0.77	1.64
10	1.68	1.56	0.95	0.99	0.75	0.50	0.45	0.21	0.29	0.41	0.00	1.28	2.3
11	1.43	1.73	1.70	0.95	0.81	0.42	0.41	0.20	0.52	0.42	0.00	1.61	1.71
12	1.71	1.82	1.70	0.91	0.93	0.41	0.39	0.24	0.25	0.45	0.00	2.34	1.8
13	1.98	1.70	2.55	0.90	1.30	0.41	0.37	0.20	0.24	0.55	0.00	3.61	1.86
14	1.93	1.63	1.72	0.96	1.12	0.42	0.36	0.70	0.20	0.49	0.00	2.84	1.9
15	2.28	1.49	1.33	1.06	0.96	0.46	0.39	0.42	0.24	0.75	0.00	2.43	1.71
16	2.37	1.55	1.12	1.24	0.96	0.44	0.38	0.77	0.22	0.76	0.00	1.89	1.7
17	2.19	1.55	1.43	1.13	0.69	0.48	0.73	0.41	0.25	0.59	0.00	1.72	
18	1.80	1.36	1.22	1.13	0.96	0.44	0.34	0.44	0.23	0.45	0.00	1.93	
19	1.70	1.82	1.62	1.04	0.89	0.44	0.35	1.37	0.23	0.44	0.00	2.83	
20	1.78	1.73	1.39	1.05	1.13	0.50	0.53	0.47	0.25	0.43	0.00	2.64	
21	1.78	1.96	1.39	1.04	1.10	0.42	0.53	0.47	0.22	0.54	0.00	1.61	
22	1.67	2.76	1.34	0.78	0.96	0.42	0.56	0.49	0.38	0.50	0.21	1.66	
23	1.93	2.40	1.57	0.89	0.82	0.48	0.55	0.23	0.32	0.50	0.25	1.52	
24	2.34	3.89	1.01	1.04	0.75	0.50	0.50	0.20	0.39	0.50	0.21	1.43	
25	2.17	2.83	1.41	0.72	0.72	0.46	0.41	0.23	0.45	0.47	0.22	1.47	
26	2.18	2.69	1.13	0.75	0.78	0.44	0.41	0.30	0.29	0.44	0.25	1.41	
27	1.90	2.28	1.16	1.00	2.23	0.42	0.41	0.24	0.00	0.42	0.22	1.46	
28	1.90	1.63	1.12	0.69	0.85	0.44	0.39	0.24	0.00	0.49	0.24	1.36	
29	1.90	1.39	0.88	0.72	0.72	0.39	0.46	0.46	0.00	0.45		2.24	
30	1.88	0.93	1.16	0.89	0.46	0.41	0.42	0.51	0.00	0.42		1.50	
31		0.72		0.78	0.50		0.42		0.00	0.40		1.42	

The amount of coagulant used and the distribution throughout the year for the clarification of the Missouri River water is shown in Table VIII.

Upon platting the few determinations of the amount of sediment contained in the Missouri River water after about twenty-four hours' natural sedimentation and the corresponding amounts of sulphate of alumina used in the clarification of the river-water as expressed in Table II and VII, the amounts of sulphate of alumina for different degrees of turbidity may be roughly approximated as follows:

TABLE IX.

Suspended matter in Missouri River water after about 24 hours' natural subsidence. Parts per million.	Amount of sulphate of alumina required for clarification. In grains per gallon.
50	0.0
100	0.5
150	1.0
200	1.5
250	1.9
300	2.4
350	2.9
400	3.4
450	3.8
500	4.3
550	4.8
600	5.3

The table indicates that no coagulant is required when the raw water contains about 50 parts per million of sediment. In this condition the water is acceptable for general use and may be even clearer than the clarified water will be found occasionally during seasons of freshets, usually from March to August, inclusive, when the turbidity of the clarified water has been found to range from 50 to 100 parts per million in a settling-basin of about four times the amount of the daily consumption. A basin of relatively less capacity requires the use of relatively more coagulant than that expressed in Table IX.

In the practical use of a coagulant it is most desirable to use a solution of constant volume but of variable strength, depending upon the turbidity of the water to be treated. The volume of the solution cannot be definitely stated, as it depends

upon the character of the apparatus used in introducing it into the water-supply and may vary somewhat with the character of the water to be treated.

The coagulant solution should be delivered under constant head into the water to be treated by some other method than that of direct-pressure pumping. It is best to provide a tank at sufficient elevation to give the requisite head of discharge and to deliver the solution into the tank by means of a pump connected with the mixing-tanks. A uniform level can be maintained in the tank by providing an overflow-pipe so connected as to admit of a return of that portion of the solution to the mixing-tank which is pumped in excess of that required for use. The delivery of the solution should be controlled by a valve suitably calibrated or a similar measuring device. A long line of iron pipe leading from the regulating-tank to the point of distribution is especially objectionable, as its capacity will speedily become seriously impaired by heavy accumulations upon the interior walls. It is far better to use brass or lead-lined pipe for the delivery of the coagulant solution. The lead is not affected by the solution and less readily clogged by deposits.

The required capacity of the coagulant apparatus depends very much upon the local conditions and volume of the water to be treated. There should be at least two mixing-tanks, each holding sufficient solution for one day's use. A tendency of the solution to stratify in the mixing-tanks should be avoided by providing and using an agitator to give the solution occasional circulation.

When weirs are employed to pass the water from one division of a settling-basin to another, the coagulant solution can be very effectively distributed through pet-cocks spaced 3 to 4 feet apart in the coagulant pipe extended along the weir. The solution discharging from the pet-cocks enters the water as it flows in a thin sheet across the weir and becomes thoroughly distributed by the circulation of the water, particularly if the water drops several inches from the lip of the weir. Fig. 22 illustrates a method of distribution through pet-cocks.

Taken as a whole, the process of water clarification by the combined process of natural sedimentation and coagulation is a

decided improvement over that of natural sedimentation alone, if for no other reason than that there ensues a greater uniformity of the general appearance of the water throughout the year. Moreover, by an intelligent use of a coagulant a rapid transition from a slightly turbid water to one of intense turbidity can be prevented.

The methodical use of a coagulant to promote and accelerate sedimentation arose at first simply because plain or natural sub-

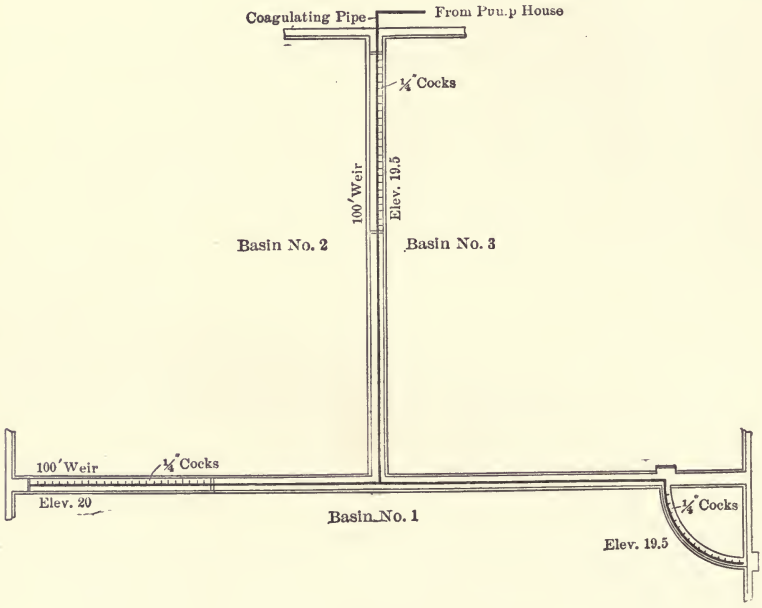


FIG. 22.

sidence failed to give satisfactory results. A way was thus opened of improving the water-supply without sacrificing property already constructed and serviceable. It was a logical step in a fuller development of the usefulness of the settling-basin, but nevertheless not a concluding step in the art of water clarification. The process fails to remove all the natural turbidity of a river-water and leaves suspended in the partially clarified water a portion of the decomposed coagulant. Even a sample of water from the clear-water division of the large settling-basin wherein objects several feet under water are discernible will

upon standing a few hours disclose small clots of the hydrate of alumina.

Convection currents, which are usual in the spring and fall of the year in a large settling-basin, never fail to cause an upheaval of the hydrate of alumina. On such occasions, when subsidence is sluggish, the amount of hydrate appearing in the clear-water basin is considerably increased.

Although there is no substantial evidence that the comparatively small amount of the decomposed coagulant which thus reaches the water-supply is deleterious, still it is desirable that all of the suspended products of coagulation should be removed from the water before reaching consumers. This is all the more important because the particles of the coagulant which thus pass the clarification works collect in the water-pipes of the distributing system where the consumption is usually small and occasionally receive a complete stirring up when there is a sudden and abnormal draft. On such occasions the consumers are liable for a brief period to receive water quite heavily charged with the hydrate of alumina.

The author noticing this experimented a little by drinking every third day for a period of about a month a small goblet (100 c.c.) of clarified Missouri River water immediately after treatment with sulphate of alumina solution in proportions varying from 11.6 to 23.2 grains per gallon with no noticeable effect. The only precaution observed was that the dose of coagulant was somewhat less than the coagulant decomposing capacity of the river-water. The experiment was simply made to fortify a position to meet any criticisms which might honestly arise on the part of any consumer who might question the safety of using the coagulant. The occasion never arose, however, to advance the experimental argument, as the only consumer who seriously objected to the use of the coagulant filed his protest in midwinter when the river was ice-bound, accompanied by the statement that the "alum was so thick he could taste it." This objection was easily met by showing him the record that no coagulant whatever had been used for several weeks preceding this discovery. This mistake was attributed to the fact

that the midwinter water of the Missouri River is more than double the hardness of the summer flow.

The experiment probably possesses little or no scientific value in the absence of medical supervision and the brief period covered, even though the dose was concentrated, but it may be used to allay apprehension founded solely on prejudice, and in this light only can it be regarded as of value.

The remarks thus far regarding the use of a coagulant apply specifically to sulphate of alumina. But so far as the principle of coagulation is concerned the remarks will apply to any coagulant. It is necessary, however, to refer specifically to another coagulant, the sulphate of iron, which is now used successfully to a considerable extent. The coagulating property of iron in a hydrated condition has long been known and has been frequently used in the purification of sewage. Lately the cost of manufacture has been so reduced as a by-product of iron works that its market value is considerably less than that of sulphate of alumina and accordingly its sale is being pushed to the front by commercial interests as a coagulant. The action of both coagulants is similar in the regard that both require the presence of carbonate of lime in the water as a base for the chemical reaction which forms the hydrate acting as the coagulant to mechanically collect the sediment and carry it to the bottom of the settling-basin in small masses. They differ, however, in the regard that the hydrate of iron is soluble in water containing free carbonic acid, while the hydrate of alumina is not soluble in the acid.

Ordinarily the carbonate of lime in natural water that is available for chemical action with the sulphate of iron is there because of the presence of an excess of carbonic acid naturally present in river-water. Moreover, one of the results of the chemical union of this sulphate of iron with the natural carbonate of lime in water is free carbonic acid, thereby increasing the capacity of water to dissolve hydrate of iron.

It is not therefore practicable to use the sulphate of iron solution alone, because after the resultant chemical reaction a portion of the hydrate of iron is dissolved by the carbonic acid and remains in solution in the water-supply. Later, if present in considerable amount, the iron gradually oxidizes and deposits

on the walls of the water-pipes, in the service-pipes, or adheres to the surface of toilet fixtures and utensils of domestic use, or blows out through a service-pipe after a period of disuse as a rusty sediment.

This difficulty is avoided in practice by treating the river-water with a solution of ordinary lime-water to neutralize the carbonic acid sometimes before and sometimes after the solution of sulphate of iron is introduced. The formation of hydrate of iron is then complete because all free carbonic acid is neutralized by the lime-water and thereby the hydrate of iron is enabled to act to its fullest extent in coagulating the sediment.

The practice of introducing lime into the river-water between the river intake and the pumps has its objection inasmuch as particles of lime collect around the valves of the pumps and may impair their operation. In one instance an engine was temporarily disabled by the accretions of the undissolved lime around the valves and in the valve-chambers.

The presence of dissolved hydrate of iron in the water-supply may appear a small matter, knowing that iron in water for drinking is frequently considered beneficial, but the tenacity with which it attaches itself to toilet fixtures and domestic utensils renders it very much of a nuisance about the household, so much so that when naturally present in a water-supply in considerable amounts various communities have sought relief by constructing purification works to remove the iron. This objectionable feature in the use of iron as a coagulant must be regarded and provisions made to guard against it in any clarification works where it is used.

The further fact that the water reaches the consumer with all free carbonic acid neutralized by the use of lime is in a measure an objection which is somewhat offset by the fact that the water may be somewhat softened as a direct result of the lime treatment. If an overdose of lime is given the water, the supply becomes flat and insipid to the taste aside from the caustic properties thus communicated to the water.

It follows that the economical and safe use of sulphate of iron as a coagulant must be accompanied by a knowledge of the character of the water to be treated obtained by making a daily

partial analysis of the water in the same manner and to the same degree that is necessary for the proper use of any other coagulant. The equipment of a small laboratory at the purification works for this purpose is a matter of small expense, and the necessary skill to make the test can soon be acquired by an intelligent attendant under the guidance of an experienced adviser to prepare the chemical reagents and to direct the method of making the daily tests. This work to be carried out successfully must become a part of the daily routine quite as much as the firing of a boiler or the oiling of an engine. It should become part and parcel of an organized management possessing sufficient practical experience to appreciate the value and absolute necessity of proper attendance to technical details.

Both the sulphate of alumina and the sulphate of iron add to the permanent hardness of the water treated, but usually to a comparatively small degree. Both are available for coagulating purposes, weight for weight, in proportion as each contributes to the formation of available undissolved hydrate of the respective bases of aluminum and iron. Consideration of the relative cost is not to be based solely upon the market value of either salt, but should also embrace the cost of the lime needed for use in connection with the iron sulphate and the cost of maintaining the apparatus and applying the several solutions.

There is no reason to suspect that the hydrate of iron will entirely subside in passing through the several divisions of the settling-basin, nor that it will be sustained in a state of suspension by convection currents to any less extent than is the hydrate of alumina or the hydrate of iron in natural ground-waters. Some of the hydrate may be expected to reach the distributing-pipes and the consumers. This circumstance need be no more objectionable with one coagulant than with another so long as iron is not present in the water used for domestic purposes to an extent to color articles of white polished surfaces, cooking utensils, or to discolor tea, coffee, or articles of food cooked with it.

A solution of sulphate of iron can be introduced with the most economy after the raw river-water has passed through a period of natural sedimentation unless the current of the river is so sluggish as to be unable to carry in suspension anything but

light sediment. The wide variation of velocity of flow between a flood and a low-water stage of a river marks a similar variation in the sediment-bearing capacity of the river, consequently it is advisable to provide for a period of natural sedimentation in the construction of any settling-basin, even though it may be unnecessary for operation the whole of each year, or prepare to use a large amount of coagulant for brief periods.

The conditions under which the process of coagulation must be applied are so similar, regardless of the particular coagulant employed, that the design of the settling-basin and the method of operating it remain substantially the same. Even the apparatus for mixing and applying the coagulant may be very similar except that for the use of the sulphate of iron and its accompanying solution of lime-water a somewhat larger apparatus is required.

Experience has so thoroughly demonstrated the need of a coagulant in clarifying muddy river-water that even a much more extended discussion of the subject seems warranted than space here admits. An endeavor has been made to touch upon the more important considerations in the belief that they will yet receive fuller development in the practice of water clarification even with the independent use of the settling-basin.

As matters now stand in the application of the process of coagulation to expedite sedimentation the least that can be said of the settling-basin is that it serves to remove the grosser impurities of muddy water and thereby prepares the water for more complete purification by some more refined process-like filtration. The most that can be said now of the settling-basin is that when constructed of a capacity several times the average daily consumption of water it can clarify a muddy water to a degree that renders it of acceptable appearance generally, but not wholly free from sediment or from the products of coagulation. It is essentially a clarifying process which thus far does not offer immunity from danger of disease germs which may reach the raw river-water. The bacterial reduction which results is purely an incident of the removal of the turbidity and is not complete. The process of sedimentation as now practiced in its most developed form can be considered safe only when applied

to the clarification of a water which when fairly well clarified is altogether wholesome.

There is a decided and natural temptation to economize in the use of a coagulant whenever there is any apparent excuse for doing so, consequently we find the coagulant used very unsystematically. Such a course is very likely to follow frequent changes of management of municipal water-works or to suggest itself to a superintendent of a private company for the purpose of increasing revenue. Very few of the small water-works under either private or municipal control seem to provide themselves with adequate facilities for either an intelligent or a systematic application of a coagulant even when experience concedes the necessity for its use. The attendants of these small plants and even of large water-works frequently get into a rut and follow the same routine day after day regardless of the necessity to modify the manipulation of the clarification works to meet the varying condition of the raw river-water. The fault of unsystematic work in this regard is not primarily that of the attendant as a rule, but rather that of the management, which may be either too inexperienced to exercise proper judgment or too far away to properly appreciate local conditions or to know actual requirements from day to day or month to month, and unwilling to delegate flexible administrative powers to the local superintendent.

Winter Treatment. — A popular impression prevails that river-water like that of the Missouri and similar rivers is most impure during an excessively muddy condition. Doubtless a river-water may contain the greatest number of bacteria at flood stage, but unsightliness and the presence of a large number of bacteria should not be confounded with dangerous pollution. It is not the sediment in the water or the actual number of bacteria which should be chiefly regarded in estimating the hygienic quality of a water, but rather the character of the bacteria found to be present, particularly when of sewage origin.

Most of our large rivers receive sewage from many towns and cities in volumes which vary between comparatively narrow limits, while the volume of discharge of a river varies between wide limits. Consequently the low-water flow of the midwinter

season dilutes the sewage to a far less degree than the flood flow of the spring and summer seasons, and accordingly we may naturally expect to detect evidence of dangerous pollution more readily during the low-water stage of winter. At this season of the year the sluggish flow of the ice-coated detrital river reduces its sediment-carrying capacity to the lowest limit and the water becomes so clear that attendants unsuspicious of its hygienic condition have no hesitation in passing it into the water-mains almost directly from the river intake. The temptation then is particularly strong to dispense with the course of treatment which the water usually receives in a more turbid condition in order to save work and to reduce operating expenses.

The self-purifying capacity of rivers and the high degree of dilution which sewage frequently receives in mixing with the water of some large rivers, even at low-water stage, may render the danger of infectious matter entering the water-supply inestimably low. But when the conditions for self-purification are unfavorable and sewage pollution is comparatively recent, the settling-basin affords an inadequate safeguard because little bacterial improvement of a polluted water can be expected from several days' storage in an ice-covered settling-basin. The treatment of the water with the ordinary coagulant at such a time can scarcely add to security by reason of the absence of germicidal properties of the ordinary coagulant and of the failure of the coagulant to collect and to settle quickly in the comparatively clear and ice-cold river-water. But it is sure that even any slight advantage that may then follow the use of the coagulant will be dispensed with for economic reasons from the moment the river begins to clear and the visual necessity for its use is no longer apparent.

It is clear, therefore, if sewage pollution can be detected in the comparatively clear water of an ice-bound river and there is available only the settling-basin in which to remove the dangerous or infectious matter, that the prompting motive of using and manipulating the settling-basin must become one directed to the purification of the water to meet a standard of hygienic purity rather than one of clearness, and accordingly in some localities *midwinter treatment must differ from summer treatment.*

Germicides.—A consideration of the use of the settling-basin for the production of a wholesome water from one that is bacterially impure presents a new and a possible range of development. The accomplishment of such a development of the settling-basin must have considerable influence upon the character and cost of purification works. It might result in the practical restriction of the biological filter, and possibly of the mechanical filter, to uses for clarification purposes and would extend the useful life of existing settling-basins indefinitely. Whenever a germicide is discovered which is harmless in its effect upon the water-supply, effective in the destruction of pathogenic germs, easy of application, and comparatively inexpensive, whether it be ozone or sulphate of copper or some other germicide, it is sure to receive an application in sterilizing water which may modify if not revolutionize our present methods of water-supply purification. Thus far the use of sulphate of copper, suggested by Dr. G. T. Moore as an algicide for use in destroying chiefly the algæ which accumulate so rapidly in storage reservoirs, promises to become a germicide of extended use just as soon as scientists, physicians, and the public can be convinced that it has no deleterious effect upon a drinking-water. Of course the use of a germicide which destroys germ life through some other effect than by a toxic effect would more quickly become popular, but experiment has not progressed far enough yet to finally determine the feasibility of the use of a germicide of the former character; but we are even now confronted with the problem of determining both the actual safety and the safe range of application of a germicide of the latter class.

In order to facilitate the use of the sulphate of copper as a germicide the manufacturers of the sulphate-of-iron coagulant design to combine the two salts in the process of manufacture so that the coagulant and germicide may be introduced simultaneously in the same solution. For certain practical reasons the use of the combined salts may be advantageous, but from a technical point of view the combination in a single solution is of doubtful utility, for the reason that the conditions requiring a variation of the coagulant from time to time do not necessarily require a similar variation of the germicide, while in midwinter

the use of the germicide alone may be needed to sterilize the water. However the manner of introducing a germicide into a water can be readily determined after the practicability and range of its use shall have been finally settled. At the present time it is too early to predict to what extent the use of a germicide may influence the use of the settling-basin, but there is now substantial warrant for stating that the settling-basin is susceptible of still greater development and that it is destined to become a more highly regarded and scientific element in the art of water purification in which germicidal action may become an important part of the process.

We cannot dispute the fact that the successful use of a coagulant and of a germicide with a view of purifying or sterilizing water forces upon us the necessity of providing skilled supervision. The character of the raw water, its influence upon chemicals introduced in the process of treatment, the preparation of the water to receive the chemicals when necessary, the specific action of the chemicals, their effect upon the water treated, and the best physical condition of the water to promote germicidal action, together with the practical difficulties which arise in treating water in large quantities, are all matters which can scarcely be entrusted to the regular attendant of water-works, but which must be understood and their worth appreciated by the management through some guiding mind trained to investigate and capable of reducing analytical data into some simple rule of guidance. We cannot avoid the necessity of treating water for hygienic purposes so long as our customs and manners of life serve to pollute directly and indiscriminately the sources from which much of our drinking-water is derived. We must either maintain our rivers in a consistent state of purity or purify poiluted water taken from the rivers to an extent conforming to a high standard of hygienic purity or seek unpolluted sources of supply.

Natural Coagulation.—The natural prejudice to the use of the ordinary coagulant in treating the water-supply and the justifiable objections to its careless use leads to suggesting a substitute which is occasionally available and is well worthy of a trial application. A few cities drawing their water-supply from

detrital rivers have also available a ground-water naturally impregnated with iron held in solution by carbonic acid. The well-known properties of an iron water to precipitate the iron as a hydrated oxide upon aeration and of hydrate of iron to act as a coagulant suggests the expediency of cities so located developing ground-water and mixing it after aeration with that portion of the water-supply which is taken from the river. Theoretically the hydrate of iron of the well-water should in part at least coagulate the remnant of the sediment in the river-water after exposure to a proper period of natural sedimentation. The use of the artificial coagulant would then be confined to an amount needed only to reinforce the natural hydrate-of-iron coagulant if for any reason it should be found to be deficient in quantity.

Of course when the amount of ground-water available is sufficient for the entire supply of the city, all that is necessary is to remove the iron from the ground-water by methods described on preceding pages. But when the amount of this ground-water is insufficient for a full supply of water the deficiency drawn from the river and naturally settled may be so thoroughly coagulated by the iron in the ground-water as to be thoroughly purified by the ordinary methods of sedimentation and filtration. Where the facilities for natural coagulation exist a trial can be so readily and inexpensively made that it is well worth the expenditure of time and money in view of the attractive inducements which the practical application of the process presents. There are localities available for the purpose where the iron of the natural ground-water has been found to vary from three-quarters of a grain to over four grains to the gallon, which is a range of iron for coagulating purposes suited to a wide range of turbidity of a naturally settled river-water. Attention should be given, of course, to the respective mineral characteristics of the water from the two sources of supply with a view of maintaining as low a degree of mineralization of the purified water as possible. But this is a matter to be worked out in detail when the application of natural coagulation is contemplated. The naturally coagulated water should be subjected to subsequent sedimentation or filtration as in the case of waters treated with an artificial coagulant.

Filtration.—The mechanical action of the process of sand filtration is a process of clogging whereby the voids of the sand gradually become filled and practically impervious near the surface of the sand-bed. In slow sand filtration the clogging process is essentially confined to a comparatively thin layer of sand at the surface and to a surface mantel of arrested impurities. A few inches of head or water pressure is sufficient to cause water to freely flow through clean filtering-sand at the ordinary rate of filtration of 45 to 135 gallons per square foot per day. But as the clogging process progresses, the water-pressure required to force water through the obstructed passage in the sand at the stated rate gradually increases until it reaches the practical limit of 4 to 6 feet. Then the surface of the filter must be relieved of the coating of impurities together with $\frac{1}{2}$ to $\frac{3}{4}$ of an inch of the surface sand where the clogging is the most intense.

The rate of filtration of mechanical filters is much greater than that of slow sand-filters, usually approximating a range of 2000 to 3000 gallons per square foot per day. This high rate of filtration causes a much deeper clogging of the filter sand, and were it not for the coagulant, which is always necessary in mechanical filtration, fine sediment would stream through the pores of the sand and enter the filter effluent. The coagulant when allowed sufficient time to act collects the particles of sediment in masses and assists materially in the operation of clogging as the water flows through the sand at the stated high rate. The penetration of clogging matter into the sand-bed is necessarily much deeper than in that of the slow sand-filter, and the rate of clogging is directly proportional to the rate of filtration. Consequently in mechanical filtration the filtering-sand cannot be cleaned by scraping, but requires nothing less than a complete stirring up and attrition of the sand, grain against grain, to separate the clogging material from the sand and to dismember the coagulated masses sufficiently to pass off in a flow of wash-water passing upwards through the sand at a rate four or five times greater than the stated rate of filtration. All of the sand in mechanical filters is washed at every washing, but only the dirty sand from slow sand-filters is washed.

Consequently if the ratio of the superficial area of mechanical filters to slow sand-filters is 1 to 25 for a given quantity of water filtered per day, then for a depth of 30 inches of sand the mechanical filter will require about twenty times more wash-water than the slow sand-filter for the same grade and amount of clogging material removed and with equally perfect mechanical facilities for performing the washing.

There is a biological action in slow sand filtration in some conditions of raw river-water which serves by vital processes of living organisms to reduce organic matter and to extinguish bacterial life. This process is more particularly applicable to waters of comparatively low sediment contents which are polluted by organic matter, particularly sewage.

The efficiency of a filter depends in a measure upon its clogging capacity, and accordingly upon its mechanical make-up and the skill bestowed in its operation and maintenance, and is measured by the degree of clearness and hygienic purity of the filter product.

Thus filtration as now understood is a refining process which as a part of a clarification process removes suspended matter which natural or plain subsidence coupled with coagulation cannot remove, or which as a purifying process removes not only the suspended matter in water but also a very large proportion of the bacteria which a raw water may contain. In the one instance the essential consideration is to clarify an otherwise wholesome water, while in the other instance it is to eliminate the living organisms with a view of excluding from the filtered water any disease germs that may be in the raw water.

With some waters the distinction between clarification and purification is a theoretical rather than a practical one from the fact that the refining process which insures *complete* clarification also accomplishes so great a reduction of bacteria that the product of the filter is considered safe and wholesome. This is true of the raw water of large detrital rivers containing a large amount of finely divided clay and possibly polluting matter in a highly diluted state.

There is this distinction, however, to be made between clarifying filtration and purifying filtration, namely, that the one is

governed largely by the appearance of the filtered water, while the other regards sanitary consideration to such a degree that the quality of the filtered water must conform to a hygienic standard of purity, which general experience, modified perhaps by local considerations, has found to be a safe guide in judging of the wholesomeness of a water.

Purifying filters must be operated continuously, whereas the operation of clarifying filters is sometimes dispensed with when the water from the source of supply is clear, as is the case with detrital rivers when ice-bound.

Notwithstanding the distinction of theory which may be made between clarifying and purifying filtration, this fact stands out clear and distinct, namely, that filtration in any form and for any purpose should be regarded as a refining process and should be so conducted that the effluent of the filter meets the requirements of a standard of purity for a wholesome drinking-water, except when applied merely to the preparation of water for some mechanical purpose, when a sanitary standard of purity may be disregarded.

In order that filtration may meet sanitary requirements the mechanical operation of filters must be under perfect control—a requirement which renders imperative a perfect control of the condition of the water admitted to the filter and of the rate of flow through the filter. Accordingly we must have a standard of hygienic purity for the filtered water and a standard, more flexible perhaps than the former, to so govern the condition of the water admitted to the filter that the filter can continually produce a water conforming with the hygienic standard.

The most available guide as to what constitutes an acceptable standard of purity of a filter effluent is the one in use in the German empire as follows:

"Section 1. In judging the quality of a filtered surface-water attention is to be paid to the following points: (*a*) The operation of the filter is to be regarded as satisfactory if the number of bacteria in the effluent does not exceed that limit which experiment has shown to be attained by good sand filtration at the water-works in question. A satisfactory effluent shall as a rule not contain more than about 100 bacteria per cubic centimeter

when it leaves the filter. (*b*) The filtrate must be clear as possible, and must not be inferior in color, taste, temperature, and chemical character to the water before filtration.

"Section 2. In order to control a water-works continuously in its bacteriological relations, it is advisable to examine the effluent of each filter daily where the conditions permit. Such a daily examination is particularly important: (*a*) after the construction of a new filter until it assumes its regular working conditions; (*b*) whenever a filter is put in operation after cleaning and for at least two days after that time, until the effluent has a satisfactory character; (*c*) after the filtration head becomes more than two-thirds of the maximum for the works in question; (*d*) when the filtration head suddenly decreases; (*e*) during all unusual conditions, especially at times of high water.

"Section 3. In order to be able to conduct bacteriological investigations according to Section 1 (*a*) the effluent of each filter must be accesseble, that samples may be taken at any desired time.

"Section 4. In order to assume a uniform system of bacteriological investigations the method given below is recommended for general use.

"Section 5. The persons intrusted with the conduct of the bacteriological investigations must furnish proof that they possess the qualifications necessary for the work. They should belong to the official operating staff itself whenever possible.

"Section 6. If a filter furnishes water which does not meet the hygienic requirements, it is to be cut out of service, provided the cause of the deficient character has not already been removed during the course of the bacteriological investigations. In case a filter furnishes an unsatisfactory effluent oftener than occasionally, it is to be placed out of service and the defects sought out and remedied.

"Section 7. In order to be able to waste poor water which does not meet the requirements each filter must be arranged so that it can be cut off from the filtered water-mains and the effluent allowed to flow away. This waste is to take place regularly, so far as the details of management permit, (*a*) immediately after the complete cleaning of the filter, and (*b*) after com-

pletely renewing the bed of sand. The superintendent in charge must determine, from his experience with the bacteriological investigations, whether a waste of effluent is necessary in each individual case after the completion of this cleaning or renewal and the length of time that must elapse before the effluent attains the required degree of purity.

"Section 8. It is necessary for satisfactory sand filtration for the surface of the filters to have ample dimensions and provide sufficient reserve to assure the velocity of filtration suitable for the local conditions and the character of the raw water.

"Section 9. Each filter-bed must be controllable as respects quantity of effluent, excess pressure, and character of effluent. It must also be arranged so that it can be completely emptied, and also be filled from below with filtered water after each cleaning.

"Section 10. The speed of filtration in each filter-bed must be capable of adaptation to the most favorable conditions for filtration at any time, and be as regular and free from sudden fluctuations or interruptions as possible. To this end the usual fluctuations caused by the varying demand for water during different portions of the day are to be equalized as far as possible by reservoirs.

"Section 11. The filter-beds are to be so arranged that their operation will not be influenced by varying levels of the water surface in the clear-water reservoir.

"Section 12. The excess filtration head must never be so great that breakage of the filtering-top layer can occur. The limit to which the excess pressure may rise without influencing the effluent is to be determined for each plant by bacteriological investigations.

"Section 13. The filters shall be constructed in such a manner that every portion of the surface of each bed shall work as uniformly as possible.

"Section 14. The walls and bottom of a filter are to be watertight, and particularly must there be no danger of an immediate connection or passage through which raw water may pass from the filter into the filtered-water main. For this purpose

special attention is to be paid to a water-tight construction and maintenance of the air-shafts of the filtered-water mains.

"Section 15. The thickness of the bed of sand shall be at least so great that the cleaning will never reduce it to less than 30 centimeters (12 inches) where the conditions permit. Nevertheless it is recommended that this minimum limit be raised to 40 centimeters (16 inches) where the conditions permit.

"Section 16. It is desired that annual reports be made from all the sand-filtration works in the German empire to the Imperial Board of Health, giving the results of operation, and particularly the bacteriological character of the water before and after filtration."

Experience or local requirements may suggest modifications of the quoted regulation. But in whatever degree modification may be found advisable, conformity thereto requires a methodical uniformity of the methods of operating filters and perfect control of the facilities for preparing the raw water for the filters.

It is well known to those who have observed the operation of filters and who have studied the results of filtration that a uniform rate of flow through the filter and sufficiently long exposure of the water to the purifying agency is necessary to insure that degree of purification which will meet the requirements of a proper standard of purity. But a uniformity of flow through the filter cannot be preserved if too wide a range of the amount of sediment in the water supplied to the filter is allowed, for then the rate of clogging becomes variable, requiring a proportionately irregular fluctuation of the filtering water-pressure even beyond permissible limits, and resulting in an irregularly turbid-water delivery of the filter. Under these circumstances the bacterial efficiency of the filter is also reduced because high bacterial variation invariably accompanies a sudden or excessive variation of turbidity of the filtered water. In other words, the filtering-machine is then overworked, overclogged, and disorganized, and accordingly fails to respond to the requirements of the situation to deliver an acceptable product.

The neglect to provide the facilities which insure a uniformity of the physical condition of the water entering a filter and of the

working régime of the filter itself accounts for numerous failures of mechanical filters and for the occasional disappointing result of slow sand-filters whenever the physical and bacterial condition of the raw river-water is made to vary through natural causes. Practice in this regard must be modified in this country and far more attention must be given to the preparation of the raw water for filtration if the results are to conform reasonably well to a standard of hygienic purity of the order quoted from the German.

So long as the theory prevails that certain forms of bacterial life in water are to be feared rather than the dissolved impurities, and so long as the science of biology, or rather the line of practice based upon the developments of this science, is unable to discriminate with precision between the harmless and dangerous bacteria and to identify individual types independently, we are bound to consider the complete removal of bacteria from the water-supply, or if complete removal be found impracticable, then to eliminate the probability and to minimize the possibility of the presence of the more dangerous forms of bacteria.

At the present time this principle or rule of practice appears fundamental, and if fundamental, then the practical standard of purity should be based upon a limited number of bacteria remaining in the filtered water rather than upon a percentage of the number in the raw water removed by filtration. But why not sterilize the water and be done with it and make this the universal standard of purity? To this puestion an answer is that it is useless to plunge into a labyrinth of practical difficulties which surely must involve one who attempts to obtain a sterile water from an artificial filter, for the filter has practical limitations depending upon its design and construction, upon the method of operation and upon the condition of the water admitted to the filter. It is perfectly plain, to use an extreme case, if the water could be sterilized before entering the filter that the work of the filter would be confined to that of clarification, and that structural and operating details would then be of a character which would prevent the filter becoming a breeding-place for bacteria. While we can scarcely hope for such perfection

in the works intended for the preparation of water for filters as to produce the result just outlined, still the nearer that sort of preparation is approached the more readily can a high standard of purification be attained. It is the pivotal point to-day in the process of water purification. If energy is concentrated upon the perfection of mechanical devices in and about filters, relying purely upon filtration as modified and influenced by the mechanical devices to produce results, the longer will the attainment of a high standard of purity of filtered water be postponed. On the other hand, if energy is devoted to the perfection of works for the preparation of water for the filters of a quality which renders high-grade filtration not only possible but positive, the sooner may we expect to see rapid advances made towards the adoption of a universal high-grade standard of hygienic purity of the water-supply. Accordingly there appears no good reason for much flexibility in the quoted standard of hygienic purity in order to meet varying natural conditions of the sources of water-supply as they change geographically, because the variable conditions call for a modification rather of the preliminary treatment of the water and accordingly a modification of the structures by means of which the preliminary work is accomplished. Whatever elasticity there may be in methods and means of water purification should be largely confined to the preliminary preparation of water before it reaches the filter. And the extent to which the physical condition of the raw water must be improved before it is admitted to the filter is largely a matter for local consideration and must be settled independently in individual instances.

The city of Bremen, Germany, takes its supply of water from the river Weser and meets the requirements of the German standard of purity of filtered water by resorting to double filtration. Eugene Goetze, Esq., chief engineer of the water-works of that city, in a paper presented to the American Society of Civil Engineers September 4, 1903, concisely outlines the physical condition of the river-water and the results of filtration in these words: "The number of bacteria in the raw water of the Weser, which normally is about 1000 to 2000 per cubic centimeter but often is only a few hundred, increases at high water,

such as occurs regularly once or twice in the fall and once or twice in the spring, to 100,000 per cubic centimeter. . . . In Bremen the water formerly filtered by single filtration contained during every flood 1000 to 3000 bacteria, instead of the permissible 100 and was quite turbid besides. Since the introduction of double filtration the city receives also, during floods, a clean filtrate with no more than about 100 bacteria per cubic centimeter."

The raw water from the river Weser is admitted directly to the Bremen filters, and the requirements of the German standard of purity are met by single filtration under ordinary conditions of the river-water and by double filtration upon those occasions when the river-water is abnormally turbid, which may be inferred is for a comparatively brief period each year.

The Bremen filters are the biological or slow sand type, which are cleaned by scraping. In this connection Mr. Goetze states: "The filtering periods between consecutive cleanings range from a minimum at a time of high water of about 7 days to a maximum of about 120 days, the average being 30 to 35 days."

It is gathered further from Mr. Goetze's tabulations of results that the abnormal turbidity of the raw Weser water scarcely reaches 200 parts per million, and that single filtration suffices for a turbidity of the raw water of about 20 to 30 parts per million or less.

In the matter of slight turbidity, or rather of a narrow range of turbidity, the few rivers of the United States which are comparable with the Weser are principally those of the New England and neighboring States. Many rivers of the Middle West and of the South are muddy, carrying a sediment in excessive amounts during the greater portion of each year. While double filtration may answer the requirements of a proper standard of purity with waters similar to that of the Hudson and Merrimac rivers, it could not be used to satisfactorily purify or clarify the waters from most of the rivers of the Middle West. The burden of sediment is far too great.

The following table taken from the report of the Water-supply Commission of St. Louis, 1902, shows approximately the average amount of sediment in river-water at several localities.

	Parts per million.
Merrimac River, Lawrence	10
Hudson River, Albany	15
Allegheny River, Pittsburg	50
Potomac River, Washington	80
Ohio River, Cincinnati	230
Ohio River, Louisville	350
Mississippi River, New Orleans	650
Mississippi River, water-works intake, St. Louis	1200
Mississippi River, Missouri side, St. Louis	1500

Elsewhere in this chapter is shown the large amount of sediment carried by the Missouri River near Kansas City, ranging from 500 to over 6000 parts per million, averaging nearly 3000 parts per million during the months of open river. When the river is ice-bound the river-water is turbid only, the turbidity ranging from 30 to 50 parts per million, or possibly less during protracted freezing weather.

It is obvious that rivers like the Missouri, Mississippi, and Ohio, having a wide range of sediment capacity marked by sudden variations, cannot receive satisfactory preparation for filtration in preliminary filters, for the burden of sediment is so great that it can be disposed of successfully by the less expensive method of sedimentation.

The settling-basin is usually the place where the roughening work of preparation can be accomplished and where the water can be prepared thoroughly for the refining work of the filter to a degree that will meet the requirements of a standard of hygienic purity of the filter effluent. But there must necessarily be some fixed limit to the turbidity of the effluent from the settling-basins, in order that the filters may satisfactorily perform the after-work. What should be this limit of turbidity?

George W. Fuller, in his report of the Cincinnati experiments relating to the "English" (slow sand) system of filtering, states: "Upon taking into consideration all of the evidence at the close of the work, it is finally concluded that the maximum permissible amount of clay in the Ohio River water applied day after day to English filters under local conditions ranges from about 30 to 70

and averages between 40 to 50 parts per million." "To make use successfully at all times of English filters it would be necessary at times of freshets to give the Ohio River at Cincinnati further preparatory treatment than is afforded by three days of plain subsidence."

Robert Spurr Weston, in his report on the New Orleans experiments, states with regard to English filters that, "barring complications from algæ growths which would tend to reduce the yield of the filter between scrapings, a silica turbidity of 35 parts per million (about 20 parts of suspended matter) would, on an average, permit according to available data regarding local conditions, a yield of about 65 million gallons of filtered water per acre, or a period of service between scrapings of about 13 days, when a head of four feet is used. . . . If the water applied to the filter should be as low in turbidity as 20 parts per million, silica standard, the yield of filtered water between cleanings would be considerably more, and the penetration of the clay into the sand layer would very likely be a little less. On an average, and in the absence of algæ growths, the data indicate that with such an applied water a yield of 100 million gallons per acre between scrapings, equivalent to a period of service of 20 days, could be obtained, and that the depth of sand removed by scrapings would be less than one inch. It seems doubtful whether, as a rule, it would be economical under local conditions to apply sufficient coagulant to reduce the silica turbidity of the filter influent to 20 parts per million (about 12 parts of suspended matter), although better results would probably accompany the lower turbidity."

Mr. Weston, in summing up the results of experiments with mechanical filters at New Orleans in purifying the Mississippi River water, gives the table reprinted on page 172.

With reference to this table Mr. Weston says: "It is believed from the above that it is not economical to apply a coagulant to reduce the turbidity of the water to below 50 parts per million (30 parts of suspended matter per million). It is also believed that considerable saving is affected by reducing the turbidity to below 75 parts per million (45 parts of suspended matter per million); but it is an open question what should be the proper turbidity between these general limits to which to reduce the

subsided water in order to effect the most satisfactory and economical filtration."

TABLE X.—APPROXIMATE RELATION BETWEEN TURBIDITY OF FILTER INFLUENT, PERCENTAGE AND COST OF WASH-WATER, AND YIELD OF THE FILTER BETWEEN THE WASHINGS.

Influent.		Wash-water.		Period of service. Hours.	Yield, million gallons per acre per day
Silica, turbidity. Parts per million.	Suspended matter. Parts per million.	Percentage.	Cost per million gallons.		
25	15	1.5	$0.27	29	150
50	30	2.0	0.36	21	110
75	45	3.0	0.54	13	70
100	60	4.0	0.72	10	50
150	90	6.5	1.17	6	30

Other statements of the general tenor may be given, but all point to what must be recognized as a fact that within proper limits the lower the turbidity of the water admitted to a filter the higher and easier becomes the permissible degree of purification of the water in passing through the filter.

Particular attention is called to the amount of wash-water that is required. It is seen in the preceding table that 6½ per cent of the total product of the filter is required for washing purposes when the raw water contains 90 parts per million of suspended matter; it is also known from general practice that the percentage of wash-water increases rapidly as the amount of suspended matter in the raw water increases, and soon reaches a point where the large percentage of wash-water coupled with an excessive use of a coagulant renders the expense prohibitive. With high sediment contents of the raw water, moreover, filters cannot be maintained in an efficient working condition.

The practical limitations to the reduction of turbidity may approximate 50 parts per million for water of silt-bearing rivers, during the unfavorable seasons of the year, by the skillful use of a coagulant after the water has experienced a period of natural subsidence of 12 to 24 hours. A greater reduction of turbidity may be readily effected during more favorable seasons of the year with the use of a less amount of coagulant skillfully applied to

an amount approximating 20 parts per million, also after a similar period of plain subsidence. But in any event the reduction of turbidity to a proper degree to admit of successful filtration of the water of silt-bearing rivers must be obtained with the aid of a coagulant used approximately in the amounts indicated elsewhere in this chapter.

Two serious difficulties arise in connection with the operation of a settling-basin in a skillful manner, one is the objection to the expense entailed in the use of a coagulant in the skillful manner and in the amounts required for the reduction of turbidity to the extent indicated above and the other is the popular objection to the use of a coagulant in large amounts. Either objection can be met only by insisting on the one hand upon a proper standard of purity of the filtered water and on the other hand by a more general familiarity with the process of coagulation and the necessity for the use of the coagulant.

The capacity of the basin that is required for coagulation and subsidence after coagulation and for temporary unserviceability during the cleaning or washing process amounts to 24 to 36 hours, which capacity added to the 12 to 24 hours' capacity required for plain subsidence makes the total capacity of a basin required for successful operation of 36 to 60 hours' supply, without considering the capacity of the needed clear-water storage-reservoir. The velocity of flow in some rivers is so low for a considerable portion of each year that the river-prism becomes a subsiding-basin. Then the raw river-water may be given a treatment with the coagulant as taken from the river and afterwards allowed a few hours' rest in a basin before going to the filters to allow the coagulant time to collect in masses large enough to be retained by the filter-sand. The current of other rivers is so rapid the year through, except when ice-bound, as to require basins of the capacity stated in the preceding part of this paragraph. Accordingly there is room for a considerable latitude of judgment in fixing upon the capacity of subsiding-basins which will meet the general requirements of local conditions most economically.

The amount of clear-water storage at the purification works need not exceed that which will safely obviate the necessity of forcing the settling-basin beyond its proper turbidity-reducing capacity.

Fig. 23 shows a plan of a water-purification works (not constructed) designed to purify water taken from the Missouri River at Fort Leavenworth, and serves to illustrate a purification works combining natural subsidence, coagulation, and mechanical filtration, and also a convenient arrangement of valves for manipulating the basin.

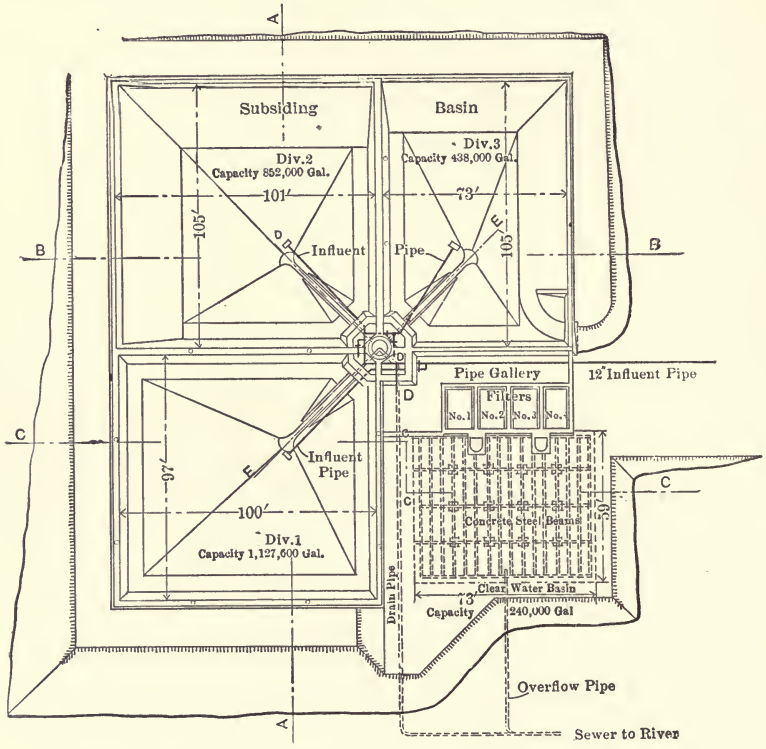


FIG. 23.—Plan of Settling-basin, Mechanical Filter, and Covered Clear-water Basin.

The purification plant is designed for the delivery of at least one million gallons per day of wholesome water.

The settling-basin is in three divisions, one division of a capacity of 1,227,600 gallons, the second of 852,000 gallons, and the third of 438,000 gallons capacity. The mechanical filters are each 17 feet long and 10 feet wide and of an estimated normal filtering capacity of about 400,000 gallons each per 24 hours at

a rate of about 100 million gallons per day per acre, or a little over 1.6 gallons per square foot per minute.

Sectional elevations of the purification works are shown in

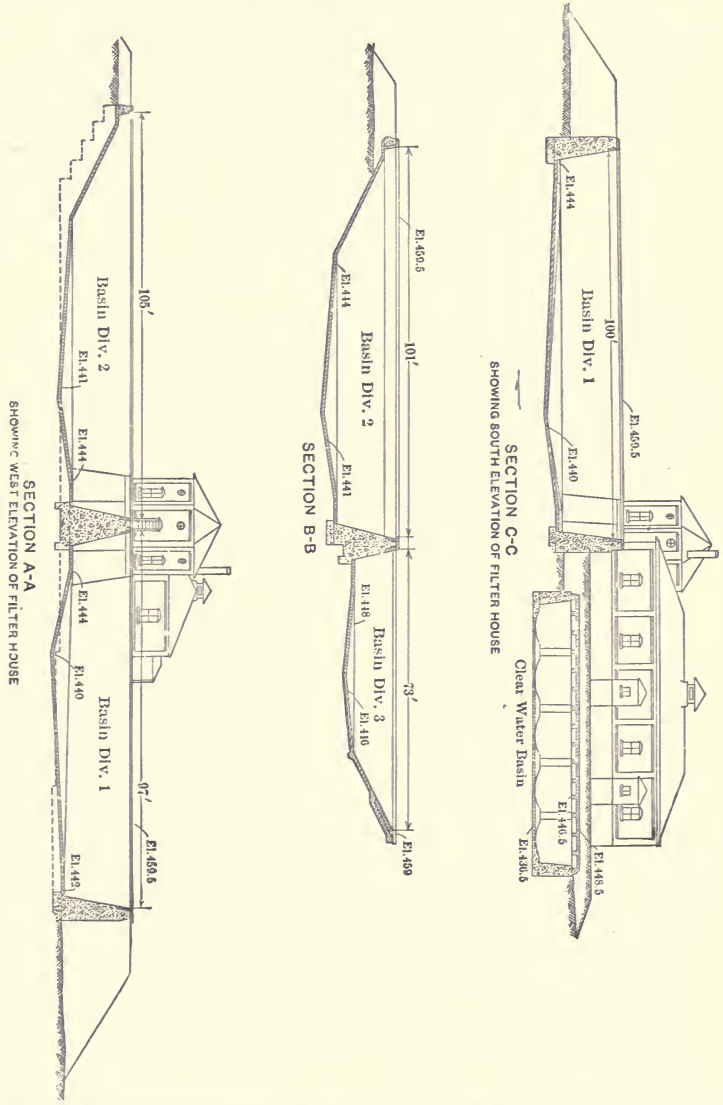


Fig. 24.

Fig. 24, and a general plan of the pipe system, valves, and gate-chamber on Fig. 25.

The 12-inch raw-water influent-pipe divides at the gate-chamber

wall into two branches, one leading into division 1, and the other into division 2 of the basin. At the end of each branch of the

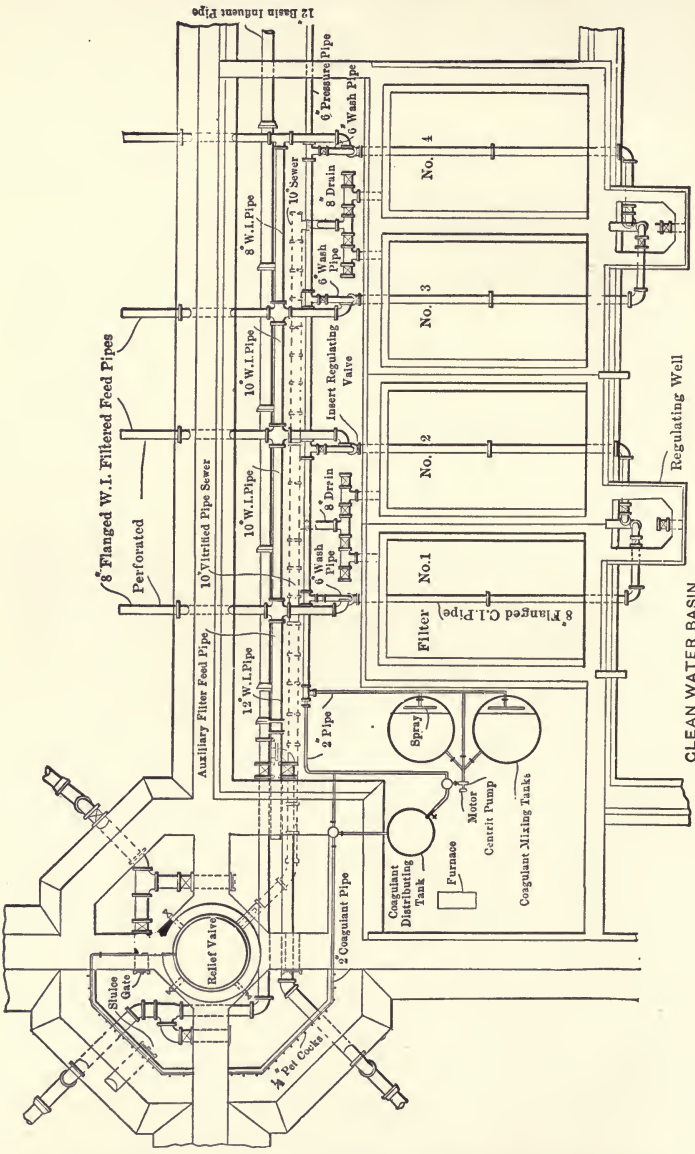


FIG. 25.—Details of Pipe Connections.

influent-pipe is a deflector which gives a horizontal radial direction to the water in a direction away from the valve-chamber.

Under normal working conditions the raw water entering the bottom of division 1 through the 12-inch influent-pipe just described, displaces an equal volume of the surface-water which

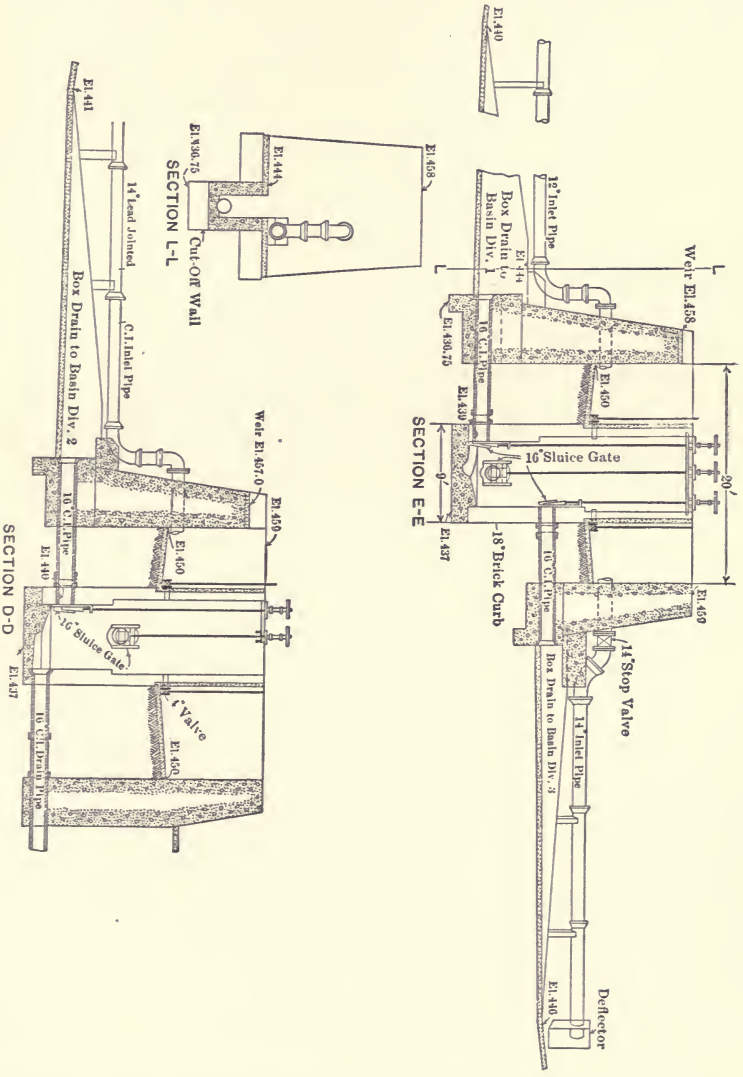


FIG. 26.

passes into the adjoining quarter division of the gate-chamber through a notch in the octagonal wall admitting of a stream about 6 inches deep and 12 feet long. In passing through this

notch the coagulant solution is introduced through a series of $\frac{1}{4}$-inch pet-cocks tapped into a 2-inch coagulant-pipe. From the quarter division of the gate-chamber the water passes into a branch pipe in the division-wall of the gate-chamber and enters

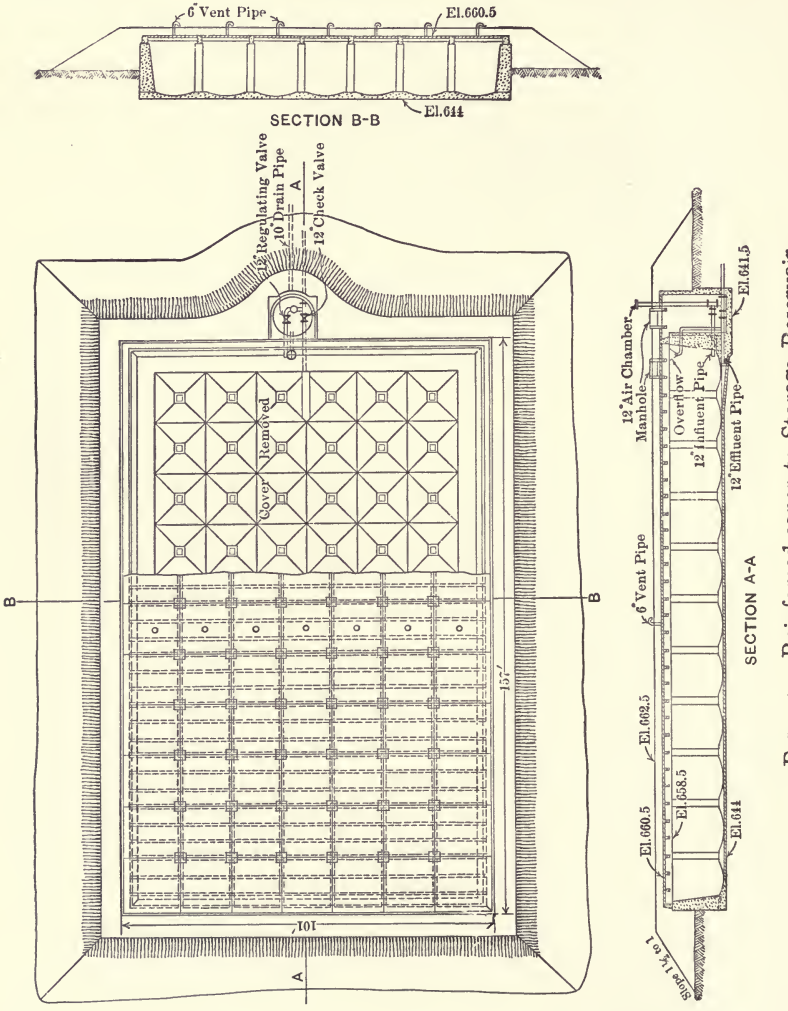


FIG. 27.—Reinforced-concrete Storage Reservoir.

the influent-pipe of division 2 of the settling-basin, which conveys it into basin 2, where it is deflected horizontally and radially as described above for division 1. The return-flow of the displaced water of division 2 enters a second quarter of the

gate-chamber by overflowing a second section of the octagonal wall, or that between the two division-walls of the basin; whence it enters through a sluice-gate the influent-pipe of division 3,

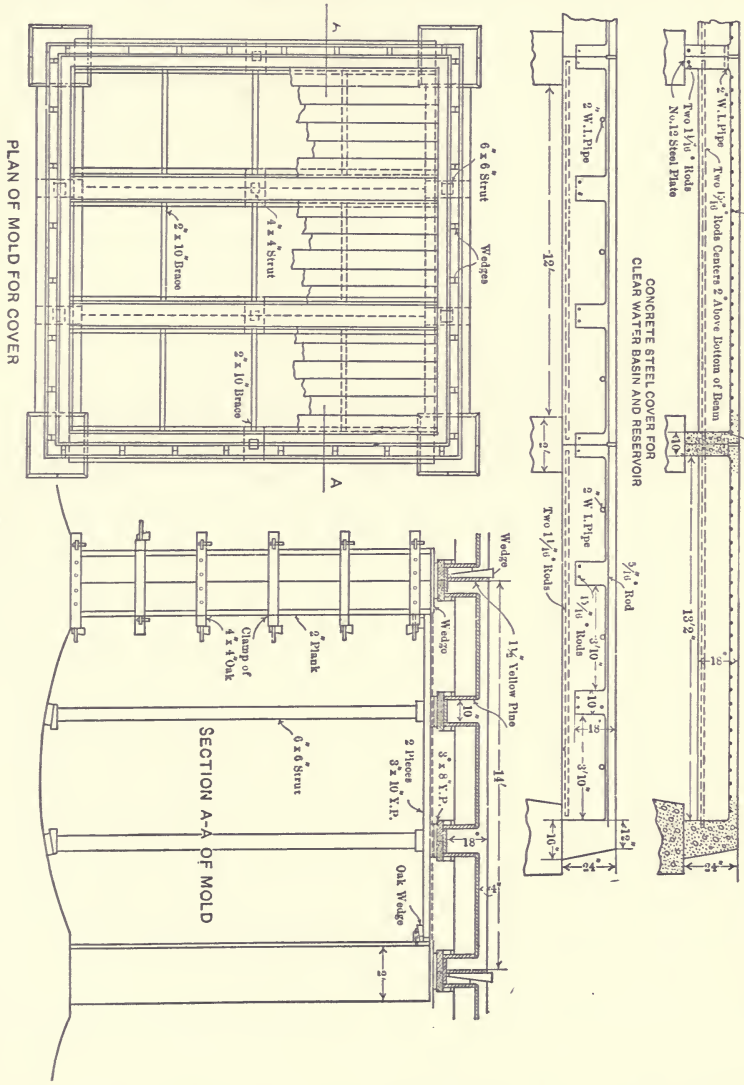


FIG. 28.

which pipe distributes the water in division 3 in a manner similar to that described for divisions 1 and 2.

The water from division 2 can be given a second treatment of

the coagulant solution in passing into the gate-chamber in the manner described for the first treatment.

When division 2 is empty for cleaning the effluent from division 1 passes from the irregular quarter of the gate-chamber into the square quarter of the same chamber through a sluice-gate, thence by pipe connection into the influent-pipe of division 3. The valve manipulation for the other two basin combinations can be readily seen from the plans.

The drain-pipe from each of the three divisions of the settling-basin, as well as the drain-pipe from each of the four divisions of the gate-chamber, enters a circular drain-well concentric with the gate-chamber (illustrated on the section elevation of the gate-chamber, Fig. 26); thence the drainage is conveyed by a 16-inch cast-iron pipe to the Missouri River.

The pipe and valve system of the filters is shown in Fig. 25. Each filter has an independent influent-pipe from division 3, also a common 12-inch influent-pipe connecting with a division of the gate-chamber for use when division 3 is being cleaned.

The plan affords such a centralization of the pipe and valve system of the entire purification works that a single attendant can readily attend to all valve manipulation under practically the same roof.

The filter-walls are of reinforced-concrete construction, as shown by the illustrations.

A distributing reservoir of 1,453,000 gallons capacity was designed to be constructed on an elevation above the barracks about one mile or more from the purification plant. A plan and section of this reservoir are shown in Fig. 27. Both the reservoir and the clear-water basin connecting with the filters are designed to be roofed with reinforced concrete resting on similarly constructed girders and pillars and covered with about two feet of earth, the roof construction being identical in both instances. The detailed drawings in Fig. 28 suffice to show the proposed manner of erecting the molds and of constructing the roof. It will be observed that the roof is designed in wholly independent panels 14 feet square, with four panels cornering on each pillar.

In passing it may not be amiss to state that the business enterprise of the men who have been interested in mechanical-filter

patents has contributed very greatly to the advancement of mechanical filtration to its present position in the art of water purification. It served to bring mechanical filtration before the public and to hold the attention of the public for years even before the merits of the mechanical filter had been thoroughly developed. Notwithstanding the efforts of mechanical-filter partisans to exploit the mechanical filter and to give it wide publicity, they often obscured the true merits of the filter in a decidedly foggy atmosphere because of their neglect to accord consideration of the technical features which successful filtration naturally embraces, and it was not until the technical features of mechanical filtration and of the mechanical devices which are necessary to insure successful operation of a mechanical filter had been thoroughly studied that it was brought to its present standard of excellence. It has been clearly shown by extended experiments and by close analytical study that mechanical filtration, like slow sand filtration, requires a thorough advance preparation of the raw water before it is in a proper condition to enter the filter. The one is intended to be as much a refining process as is the other; as good results may be expected from one as from the other; and the performance of the one method of filtration over the other is often a matter of adaptability and of cost.

It may be safely said from a purely technical standpoint that with a sufficient advance preparation of the raw river-water either method of filtration can be used in any locality, even where such waters as that derived from the Missouri, Mississippi, or Ohio River are to be purified. It is found, however, that slow sand filtration together with the necessary subsiding basins for the proper preparation of the water for these filters involves so great an expense as to be practically prohibitive; and it is further believed that slow sand filters, when used in connection with purification works which require so thorough an advanced treatment of the raw water as that of the river just named, become in themselves purely mechanical filters, for the reason that the long period of subsidence and the large amount of coagulation to which waters from such rivers must be subjected doubtless remove from the raw water the elements or properties which are needed to con-

struct the filtering mantle which is supposed to be the effective element in the operation of the slow sand *biological filter*.

In drawing this chapter to a close it may be briefly stated as a summary of what has been said that natural subsidence, coagulation, and filtration are necessary for the proper purification of the water from detrital rivers generally, and that there must be not only a high standard of purity for the effluent of a filter, but also a restricted range of turbidity to the filter-influent water. Without proper regard to the principles embraced in the three distinct processes and to the bacterial and physical condition of the water admitted to a filter it is found impossible to secure a water of uniform purity, or one that should be recognized as conforming to a proper standard for a good and wholesome water. The general practice of the present day is not up to the standard indicated. Principles are often disregarded, especially the one of preliminary preparation of the water for filtration, carried even to the extent of sometimes adding a coagulant to a water of very low turbidity in order to make filtration successful even through slow sand filters. If the author has succeeded in making this point clear and in helping on the work of reform in the directions indicated, it is believed that the chief obstacle to successful water purification by filtration can be overcome.

CHAPTER III.

PUMPING-ENGINES.

THE pumping-engine is so important a part of the equipment of many water-works as to warrant a few words regarding its use and maintenance. Little can be said regarding the manufacture of machinery of this kind, as it is a special work carried out by corporations fully equipped to design and manufacture. The manufacturers, however, desire a thorough knowledge of the service in which a pumping-engine is required to work in practice, of the source of the water-supply, and of any physical conditions which may affect the installation of the machine and the service expected of it.

Accordingly the engineer's specifications under which proposals of design, installation, and manufacture are invited and the purchase is made are usually confined to general stipulations defining and fixing the obligations of the contracting parties, a detailed statement of the conditions affecting both the installation and operation of the pumping-machinery as above outlined, and the terms of payment, etc. Frequently a certain type of machine is specified which is known to possess superior merits for a particular kind or range of service. Usually plans are submitted with the specifications to illustrate the description given therein and to show the design and arrangement of the pumping-station, together with the arrangement of plant and machinery already installed or about to be installed.

It is usually customary for the purchaser to build the foundations upon which the machinery is to rest after plans of the manufacturer showing the anchor-bolt lay-out and the design of the super-

structure to accommodate the type of machine proposed for installation. Such a division of the work serves to facilitate matters and to reduce expense.

The manufacturer, while providing the valves, fittings, and accessories necessary to properly govern and operate the machinery, is seldom expected to carry his work outside the walls of the building in which the machinery is installed.

Usually the purchaser requires of the manufacturer some sort of guarantee as to the degree of economy with which a proposed pumping-engine will perform the work of pumping water. Evidently the larger the number of gallons of water delivered by the pumps of a pumping-engine with the consumption of some given amount of fuel under the boilers or of steam by the engine, as the case may be, the less becomes the fuel expense and, relatively, the higher the economy of the machine becomes in operation. The measurable work of the pumps never accounts for all of the energy developed by the combustion of coal in the boiler-furnaces in the manufacture of steam, nor even for all of the energy in the steam admitted to the steam-cylinders of the engine actuating the pumps, for a large portion of the heat of the coal is unavoidably wasted, and also a portion of the heat energy actually absorbed by the steam is unavoidably lost in passing through the pipes leading to the engine, in the steam-cylinders themselves, and in overcoming the friction of the moving parts of the machinery which it actuates. Accordingly in practice the term "mechanical efficiency" is applied to pumping-engines, meaning thereby the relation which the work actually done by the steam in the cylinders bears to the work actually accomplished by the pumps in lifting water. Thus if the work done by the pumps divided by the indicated work in the steam-cylinder equals unity, as would be the case were the actual and the indicated work the same, then the engine is perfect mechanically and the mechanical efficiency is 100 per cent. But as this percentage measure of mechanical efficiency decreases the work of the engine decreases on the scale of economy; for instance, a machine giving a mechanical efficiency of 90 per cent is better designed and constructed than one giving 70 per cent.

But the ordinary term for a measurement of economy of a pumping-engine is "duty" in terms of foot-pounds—meaning the

number of pounds of water a pumping-engine is capable of lifting one foot high with the expenditure of a unit volume of fuel or steam. Formerly the unit volume was 100 pounds of coal or combustible. This basis of measurement is unsatisfactory to both the manufacturer and the purchaser of a pumping-engine because it necessarily embraces the evaporative capacity of the boilers and the mechanical perfection of the boiler-setting and steam-piping connecting the boilers with the pumping-engine. In other words, the pumping-engine ceases to be a unit of consideration in itself in a duty test. And from the very nature of the combination of engine, piping, boilers, and furnaces, it is sometimes difficult and embarrassing to fix responsibility for failure of an engine to give a stated duty when the several units of the combination are the workmanship of as many different contractors, Moreover the technical details to be observed and the measurements to be taken in a combined test are many and often considerably complicated.

Of late years it has been customary to use the unit volume as one thousand pounds of steam delivered to the engine at some stated pressure. The choice of this unit of volume simplifies matters, and by isolating the boiler or boilers furnishing steam for the engine under test, or by measuring the amount of steam diverted from the engine under test for use in other directions, the observations and measurements relating to a test are rendered less complicated than in the former case just described. This method of measuring the duty of a pumping-engine, while satisfactory from a purchaser's or a manufacturer's point of view, does not meet the requirements altogether for ordinary station practice, for the reason that it fails to show a direct relation between the work done in pumping water and in operating other station appurtenances and the coal consumed under the boilers. Consequently for station purposes it is often convenient to refer to economy in various terms, as steam consumption per horse-power per hour, coal consumed per horse-power per hour and coal consumed per 1000 gallons of water raised a given height, etc.

In order to facilitate computations and to compare approximately the various units of measurements the following formulæ and tables are offered:

Let D = duty of pumping-engine in foot-pounds per 1000 pounds of steam consumed by engine;

H = head or total lift of pump, including suction-lift;

V = volume of water pumped per unit of time; say, for convenience, one hour;

W = weight of unit of volume of water, 8.34 pounds per U. S. gallon;

S = weight of steam consumed per hour;

H.P. = pump horse-power. One H.P. = 33,000 ft.-lbs. of work per minute or 1,980,000 ft.-lbs. per hour;

C = coal consumed per H.P. per hour.

Thus
$$D = \frac{H \times V \times W \times 1000}{S},$$

$$\text{H.P.} = \frac{H \times V \times W}{1980000},$$

$$\frac{D}{\text{H.P.}} = \frac{1000 \times 1980000}{S},$$

or
$$\frac{S}{\text{H.P.}} = \frac{1000 \times 1980000}{D},$$

or
$$\frac{S}{\text{H.P.}} = \frac{1980}{\text{duty in million ft.-lbs.}}$$

= steam consumption per horse-power per hour.

From the last equation the following table is computed:

STEAM CONSUMPTION PER PUMP H.P. PER HOUR FOR STATED DUTIES IN MILLION FOOT-POUNDS.

10	15	20	25	30	35	40	50	60
198	132	99	79.2	66	56.6	49.5	39.6	33
70	80	90	100	110	120	130	140	150
28.3	24.7	22.0	19.8	18.0	16.5	15.2	14.1	13.5

COAL CONSUMPTION PER PUMP H.P. PER HOUR, AT AN EVAPORATION OF 10 POUNDS OF WATER PER POUND OF COAL, OR WHEN $C = S/10$ H.P., FOR STATED DUTIES IN MILLION FOOT-POUNDS.

10	15	20	25	30	35	40	50	60
19.8	13.2	9.9	7.92	6.6	5.66	4.95	3.96	3.3
70	80	90	100	110	120	130	140	150
2.83	2.47	2.2	1.98	1.8	1.65	1.52	1.41	1.35

To illustrate the use of the table, suppose that a pumping engine was found to consume 22 pounds of steam per pump horse-power per hour, as measured by the weight of water evaporated by the boiler, and also that the boiler was found to evaporate 6 pounds of water per pound of coal. The duty, then, by the table would be 90 million foot-pounds per 1000 pounds of steam or per 100 pounds of coal on the basis of an evaporation of 10 pounds of water per pound of coal, or 54 million foot-pounds per 100 pounds of coal consumed under the boiler, and the coal consumed per pump horse-power per hour would be about 3.6 pounds.

In every pumping-station a record should be kept of the amount of coal consumed in the boiler-furnaces and the number of revolutions made by each unit of the pumping-machinery and of the head or pressure against which the pumps operate, for information of this kind systematically and methodically recorded enables an engineer to keep posted upon the station duty of the several machines under his charge.

The station duty is found to vary considerably from the test duty usually embodied in specifications governing the purchase of a pumping-engine, for the station duty is usually based upon a unit volume of the coal consumed and often takes into account boiler economy, waste, and the fuel consumed for all auxiliary engines as well as for the pumping-engines actually delivering the water into the distributing pipes. For instance, in a pumping-station recently inspected it was found that one pumping-engine which on test delivered 338 gallons of water per pound of fuel actually delivered in daily service 263 gallons of water for every pound of fuel consumed, that a combination of large and small pumping-engines delivered 156 to 172 gallons of water per pound of coal, and that the average annual station duty for a period of three years was from 150 to 167 gallons of water per pound of coal consumed. It was also observed when a proper correction was made for the slips of the pumps that the station duty became reduced for one year to 124 gallons of water per pound of fuel consumed. The working head against the pumps was about 323 feet.

Thus it is observed that a test duty may be much in excess of the ordinary station duty and that when attendants become

careless or helpless regarding the mechanical condition of the pumping-engine the station duty based upon coal consumed and water actually delivered into the mains may run very low.

Attendants may find it convenient to have a table or diagram showing the station duty of the pumping-machinery per pound of fuel consumed when the head or pressure against which the pumps operated is known. A diagram of this kind may be found on page 189.

On other occasions when the annual amount of fuel to be charged against some specific unit volume of water, as 1000 gallons, is desired the diagram on page 189 may be found convenient.

Again, the annual cost of fuel per one million gallons of water raised some stated height, when the station duty is known, is another convenient unit often referred to; accordingly two diagrams on page 190 may be of service. The diagrams are based upon 300 feet head or 130 pounds gauge pressure, but a reduction to any particular working head in feet may be made by multiplying the annual fuel cost as ascertained from the diagram by the ratio

$$\frac{\text{Particular head in feet}}{300}.$$

A method of keeping the pumping-station records is given below. It may be varied somewhat to suit local requirements.

DAILY STATION RECORD OF PUMPING-ENGINES.

Date.....

Description.	Nominal capacity.	Hours of service.	Counter reading.		Gallons of water pumped.	Water pressure.	Steam pressure.	Vacuum.
			Starting	Stopping.				

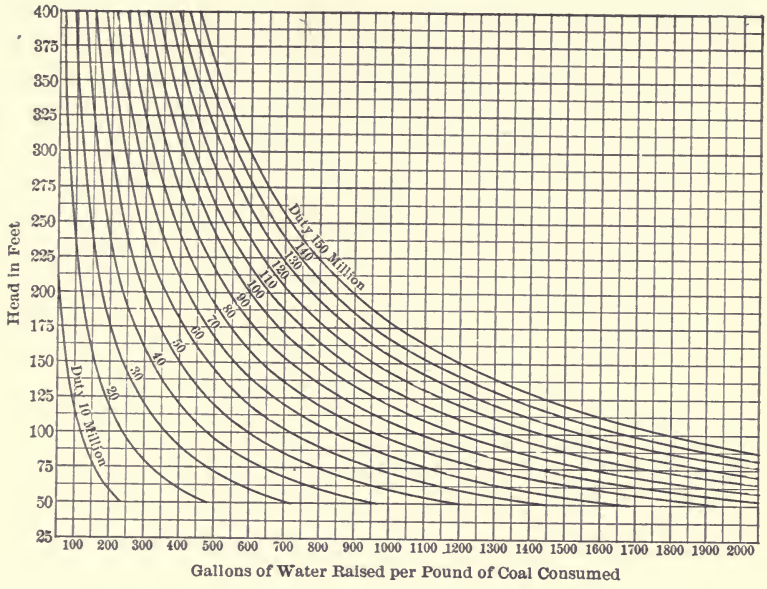


FIG. 29.

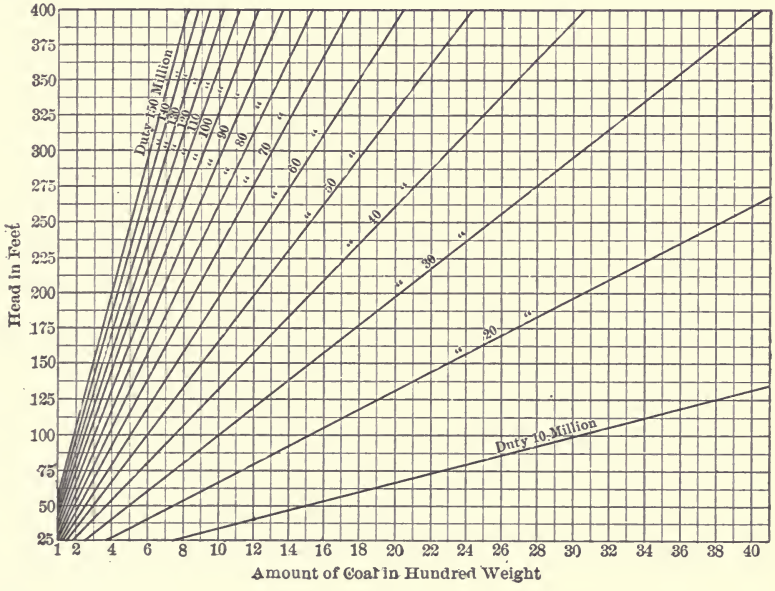


FIG. 30.—Amount of Coal Required Annually to Raise 1000 Gallons of Water Daily

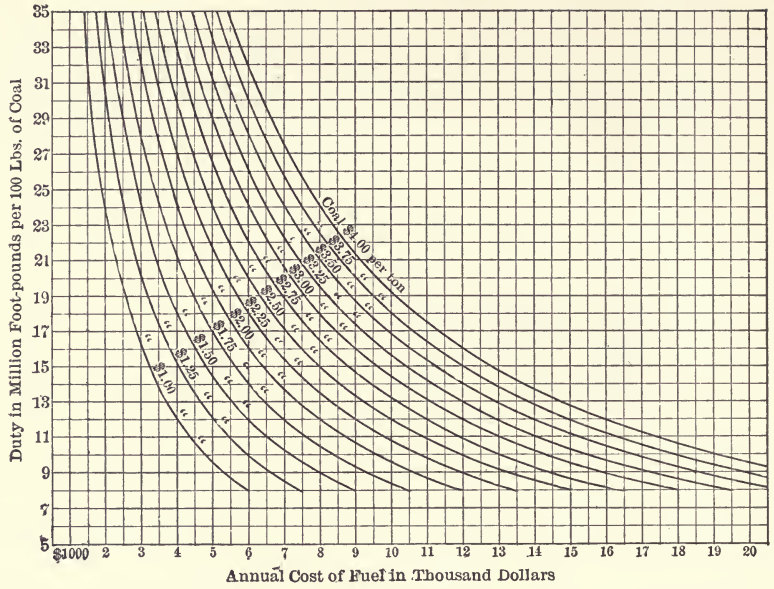


FIG. 31.—Annual Cost of Fuel Required to Raise 1,000,000 Gallons of Water Daily 300 Feet High, allowing 5% for Slip.

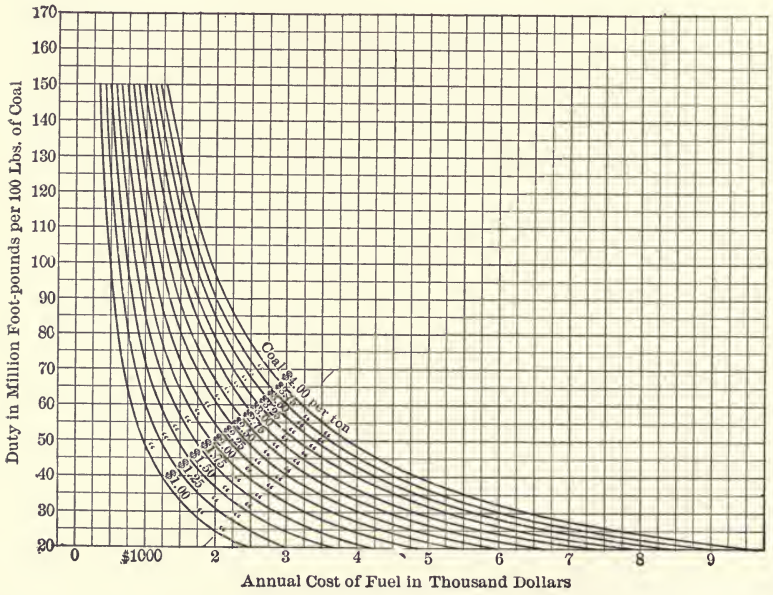


FIG. 32.—Annual Cost of Fuel Required to Raise 1,000,000 Gallons of Water Daily 300 Feet High, allowing 5% for Slip.

DAILY STATION RECORD OF BOILERS.

Date.....

Description.	Hours in service.	Steam pressure.	Temperature.		Fuel.		Remarks.
			Feed-water.	Flue-gases.	Coal.	Ashes.	

A table has been prepared based upon data collected from the annual reports of water departments for the purpose of illustrating the station duties that may be expected from various types of pumping-engines.

TABLE XI.

Average of years.	No. of pumping days.	Coal consumed in raising steam in per cent of total coal consumed.	Average head against pump.	Gallons of water pumped per pound of coal.	Pounds of coal required to raise 1000 gallons of water to stated height.	Annual engine duty per 100 pounds of coal consumed.	Remarks.
1			100	190	5.3	15786000	Ordinary duplex comp. condensing
10	100	0.51	176.5	152	6.64	22370000	
1		6.08	173.2	288	3.48		High-grade comp. cond. duplex
11	302.4	6.33	242	259.3	3.86	52273000	Holly quad.
10	126.6	2.82	181	335.7	2.982	50725500	Cornish beam
1			152.8	406.8	2.458	51843600	Rotative duplex comp. condensing
1	365		61.69	638.2	1.56	32807533	do.
1			52.5	1162.5	0.86	50870200	Vert. marine type
7	360.6	0.26	184.89	792.52	1.264	122076700	High-grade vert.

It will be observed from the preceding table that the head against which a pumping-engine works is an important factor in comparing economy of operation and that if one is to draw fair comparison the work of the engines should be based upon a common head of say 100 feet. Making this reduction we find the pounds

of coal required to raise 1000 gallons of water 100 feet high for the several types of engines referred to are respectively as follows: 5.3, 3.76, 2.01, 1.59, 1.65, 1.61, 2.53, 1.64, 0.68.

The above table and succeeding computations should not be accepted as truly comparable, for unless all the conditions of service are known in each instance and an allowance made accordingly unfair comparison may result. All that is desired by the illustration is an approximate comparison of the several types of engine. For instance the same type of engine working in direct-pressure service would not give as high a duty as when working under the conditions of reservoir service for the reason that in the former instance the speed of the engine fluctuates with the momentary and hourly requirements of service, while in the latter case the engine may operate under full load at its most economical speed hour after hour and day after day, regardless of the rate of water consumption.

It is not easy to obtain pumping-station duties of the small water-works which use the non-condensing duplex type of pumping-engines for the reason that in the majority of cases the records are incomplete.

The slip of pumps embraces the water which in the operation of a pumping-engine escapes through leaky or defective valves or churns backward and forth through a defective plunger packing; that is to say, it is that portion of the pump-plunger displacement which does not reach the distributing system although raised to a pressure corresponding to that of the portion which does enter the distributing main. It requires proportionately as much fuel to support the slip of a pump as it does the delivery of the pump. For instance, to take an exaggerated case, if the slip of the pump is 50 per cent, then one-half the fuel consumed in running the pumping-engine would be used in supporting slip and one-half in delivering water into the force main, or the annual cost of fuel would be nearly double what it should be were the pumps maintained in an efficient condition. It is surprising to see the inefficient condition which pumping-machinery is sometimes allowed to reach. To illustrate this condition reference is made to a controversy which arose a few years ago between a municipal corporation owning its own water-works and a private corporation dependent

upon the former corporation for its supply of water delivered to the company's distributing-pumps at a very low pressure. The controversy of three years' standing related to the actual delivery of a pumping-engine of 9 or 10 million gallons capacity per 24 hours for the stated period of years and involved a considerable sum of money. A commission was finally appointed to arrive at the facts and to settle the controversy.

Three independent methods of measurement were finally adopted by the commission:

1. The plunger-displacement method, with the pump operating against a tightly closed gate-valve on the force main. This method of slip measurement was acceptable to the owners of the machine and the purchaser of the water, but discarded by the party selling the water.

2. The piezometer method, by means of which a series of hydraulic grade lines were determined on the very low-pressure flow-line furnishing water to both parties, into which a measurable and known quantity of water was delivered. The hydraulic gradients were established both above and below the point from which the volume of water in dispute was abstracted.

3. A weir measurement of the actual amount of water delivered by the pumping-engine in question when operated at different speeds under the pressure conditions corresponding to regular service. The difference between the nominal delivery of the pumps based upon plunger displacement and the actual delivery as measured by the weir gave the slip.

The slip of the pumps as measured by these totally different and independent methods is as follows:

1. Displacement method.....	2872.34	gallons per minute
2. Piezometer method.....	3090.00	" " "
3. Weir method.....	2926.46	" " "
Average of the three methods..	2962.9	gallons per minute

The average computation of the slip was accepted as correct, and the test proved the substantial accuracy of the displacement method of slip measurement, and enabled the commission to employ various similar measurements of slip, previously taken, in

the construction of a diagram used in the correction of the records of the three preceding years and to arrive at the actual delivery, which was found to aggregate nearly 1960 million gallons of water and the slip to aggregate nearly as much.

The condition of the pumps at the time of the test is represented by the following table based upon weir measurement.

Test number.	Revolutions per minute.	Nominal delivery in gallons per minute.	Percentage which actual delivery was of nominal delivery.
1	8.8	2702.79	0.0
2	10.0	3102.47	4.5
3	10.6	3338.09	11.2
4	11.2	3507.12	15.5
5	12.0	3786.97	21.8
6	12.8	4059.35	27.0
7	14.13	4412.05	32.8
8	14.5	4543.70	38.8
9	15.4	4702.51	36.9
10	16.8	5309.22	44.2
11	18.2	5751.66	48.5

It is obvious from the table that the fuel required to raise the 2702.79 gallons per minute of water to the pressure of distribution was absolutely wasted. This glaring example of waste through slip serves to illustrate forcibly the importance of timely repairs of the moving parts of the pumps.

Moreover the comparative tests show that the displacement method of measuring slip by operating against a closed valve on the force-main under the pressure conditions of service is sufficiently accurate for practical purposes. It has the further value of being applied in any pumping-station with little or no expense.

The valves in the force and suction chambers of the pumps become the source of serious slip whenever the valve-seats are allowed to become scored or the rubber disks unusually worn, and either one of these two causes will lead to the presence of the other. Usually the diameter of the pump-valves is about $3\frac{1}{2}$ inches, and unless the rubber disks are capped with a metal plate of somewhat smaller diameter they open over the radial arm of the valve-seats under heavy pressure and allow the escape of water at each stroke of the pump. Prolonged leakage of this character under high pressure eventually scores the valve-seats and cuts the valve-

disks irregularly—so much so that the hissing sound of water escaping through the valves is often distinctly audible.

The lift of the valves should never be much, for the wear and tear of pump-valves depends not only upon the number of reversals of the pump-plunger, but also upon impact at the instant the direction of the stroke of the plunger reverses. The impact becomes most serious when the pumps are forced beyond their rated capacity.

Little need be said about the size and lift of pump-valves, as the purchaser can scarcely define details of this character precisely without assuming responsibility, in a measure at least, for the design of the machine and proportionately for the results of operation. These details are usually left to the judgment of the manufacturer under reasonable guarantees of maintenance and renewals of broken or defective parts for some specific period of time. Generally speaking this course of procedure duly protects the interests of both the purchaser and the manufacturer if the purchaser's specifications define with proper precision the actual conditions of service. If these conditions are not so defined, it is scarcely conceivable that the interests of the purchaser would be better or more safely served by interfering with the details of design except through the instrumentality of a master of the art of design.

Loss by slip through the plunger packing is much more liable to be neglected in pumps internally packed than in those externally packed, for the reason simply that any lack of vigilance on the part of an attendant is less apparent in the former than in the latter case and correspondingly less liable to correction by an overseer, and for the additional reason that many inside-packed pumps are metal-packed and cannot be repaired except by lathework, for which class of work most small water-works are not prepared. Accordingly wear is frequently allowed to progress to an extent which under more favorable circumstances for repairs and better facilities for packing would not be tolerated.

It might be observed that inside metal-packed pumps are only adapted to the pumping of water entirely free from dirt, and even when used in pumping clear water under moderate pressure it is profitable to make a timely purchase of duplicate plunger and rings to replace the old ones as they become worn sufficiently to

admit of pronounced slip. The old ones can always be repaired in some near-by city if no facilities are available at home for the purpose.

The lowest-grade pumping-engine on the score of economy that is permissible to install in a water-works for daily service is the compound, non-condensing, duplex pumping-engine. The low economy of this class of machinery is offset in a great measure by its low cost and simplicity of design and operation, which makes it popular with small communities for the first five to ten years of experience with water-works. In fact it is peculiarly adapted to the conditions and circumstances surrounding the introduction of water-works in small towns, because its simplicity of construction renders it the least likely of disarrangement under the direction and supervision of a very ordinary mechanic and occasions the least expense for maintenance and repairs, and because economy of operation is of small consideration for the very few hours of service daily during the probationary period which serves to render a community acquainted with the convenience, nature, and value of a public water-service, and to accumulate the experience and the proper appreciation of essentials that are necessary in the successful management of a water-works. An investment in more expensive and more complicated machinery than the simple duplex machine would sometimes prove unprofitable until proper experience had been acquired and the water-service had become extended and popular.

Generally speaking, the inside metal-packed plunger-pump is preferred, and usually installed during the probationary period, for water-pressure under 150 pounds per square inch. Fibrous packing, sometimes substituted for metal plunger-packing, adds to the frictional resistance of the plunger in operation and accordingly reduces the mechanical efficiency of the machine.

For pressures above 150 pounds per square inch it is found economical and much safer to approximate as nearly as possible a cylindrical form of construction of the pump-chambers, and accordingly the usual form of design is four cylindrically formed pump-chambers standing upright in pairs, and with a plunger working through two outside stuffing-boxes for each pair of pump-cylinders, as illustrated by Plate XV, page 103. These

stuffing-boxes are fibrous-packed, usually with braided Italian hemp, and show any slip or leakage of water around the plunger. The double fibrous-packed plunger offers more than twice the resistance of the single inside metal-packed plunger previously described. However, this increased friction is unavoidable in high-pressure service and under careful mechanical supervision it need not be a large percentage of the total frictional resistance, which in high-pressure work is largely the pipe resistance of the force-main and distributing pipes.

The inside-packed plunger possesses the further advantage of being able to sustain a higher vacuum on the pump suction-chamber than the outside-packed plunger, because the latter is continuously exposed to atmospheric pressure and will allow air to enter the suction-chamber of the pumps through an imperfectly packed plunger stuffing-box gland.

The duplex pumps will often short-stroke; that is to say, they will sometimes run with a shorter stroke than that necessary to a full-capacity delivery of water. It is therefore customary to have a stroke-adjustment mechanism actuated by hand-wheels communicating by a spindle with the interior of the engine, by means of which an adjustment can be made to correspond with full-stroke delivery as soon as the engine after starting is well warmed and the pump is under full load.

When the pumping-engine is working under heavy pressure it is a wise precaution to have attached to the steam-valve of the engine a water-pressure governor like the Fisher governor, which shuts off the steam upon a sudden fall of water-pressure like the bursting of the force-main or similar accident.

The conditions of service in many towns require stand-pipe pressure for domestic service and direct pressure for fire service. In the one case the stand-pipe communicates with the distributing system of pipes, absorbs whatever water comes from the pumps in excess of the consumption and accordingly governs the pressure upon the pipe system; in the other case the stand-pipe is valved off from the distributing pipes and no more water can be delivered into the pipe system than that which is drawn out of it by consumption through the house service-pipes or fire-hydrants. Accordingly in direct-pressure service the pressure upon the pipe

system may be increased through the operation of the boilers and pumps much above the ordinary stand-pipe pressure. But a compound duplex pumping-engine designed to work economically under a stand-pipe pressure cannot work in a similar manner under fire-service. This difficulty is overcome by providing a valve which when opened admits high-pressure steam into the low-pressure cylinder ordinarily using steam expansively, thereby converting the machine into a simple engine using steam altogether at boiler-pressure. For the time being all concern to secure station economy in the use of fuel is abandoned in an effort to furnish prompt and efficient fire-service, and justifiably so, for the period of fire-service, always comparatively brief, is one which demands the protection of property exposed to destruction at any pumping-station expense of operation. A similar adjustment of rotative pumping-engines to the requirements of a variable service is made by means of an adjustable cut-off.

It is usually customary to place a check-valve on the force-main near the pumps, but with doubtful propriety in small water-works when accompanied by a gate-valve, for the reason that a check-valve does not relieve a pump of water-hammer and is unnecessary to keep the pressure off of the pump-valves when the engine is not working, as it takes but a few minutes to close a stop-valve of 12 inches and less diameter; with water-works having large water-mains the situation is so entirely different that a multiple check-valve is indispensable. The pounding of the check may be a very disagreeable if not a dangerous feature usually experienced when pumps reach or somewhat exceed their rated capacity.

A long force-main connecting the pumps with the distributing system is frequently exposed to a heavy water-ram, but it can be greatly relieved from the shock of the surging water by a relief-valve located usually in the pumping-station. Under some conditions of service—where, for instance, a heavy static lift is required in order to reach a high elevation, and in addition where there is the resistance of a long force-main to overcome—a relief valve is absolutely essential, particularly when the pipe-lines are small.

A grade of pumping-engine next above that just described is the compound condensing duplex type. In this type of engine the

low-pressure cylinder is somewhat larger in proportion to the high-pressure cylinder than it is in the type previously described, thereby admitting of greater benefit of steam used expansively. The exhaust-steam passes into a condenser, where it is condensed and subsequently returned to the boiler or wasted, as the case may be. Local conditions usually decide whether a surface or a jet condenser can be most economically installed in connection with the pumping-engine. The saving of fuel through the higher expansion of steam in this type of pumping-engine and in the use of a condenser should increase the station duty fully 25 to 30 per cent.

A second step in advance is the installation of a triple-expansion duplex or a horizontal compound fly-wheel pumping-engine of the condensing kind. Small units of either type of engine are now manufactured for small water-works where the average daily consumption does not exceed one-half million gallons of water, which the manufacturer will guarantee to give a test duty respectively of 70 or 80 million pounds of water raised one foot high with the expenditure of 1000 pounds of steam. These types of engines of the small units stated should give a station duty more than double that of the type of engines first described.

In installations of large units the test duty will run over a hundred million pounds of water one foot high with an expenditure of 1000 pounds of steam, and the station duty becomes proportionately as high.

Duties as high as 130 to 160 million pounds of water raised one foot high with the expenditure of 1000 pounds of steam may be obtained from double- or triple-expansion fly-wheel engines in pumping large volumes of water to a considerable height. Engines of this type are adapted for use in connection with the water-works of large cities which depend upon the constant use of the pumps for their water-service. They are expensive machines and require skilled mechanics as attendants. Very high station duties may be maintained with this type of engine working under favorable conditions, and very long life may be expected of massive, slow-moving, and self-contained pumping-engines of this type.

The centrifugal or turbine pump actuated by a reciprocating engine is peculiarly adapted to low-lift work because of the cheapness with which such machinery can be installed in comparison with

economically working reciprocating pumping-engines. Installations of this character are frequently recommended in connection with settling-basins and filters and numerous other uses of a similar kind. A few installations are to be found of high-lift turbine-pumps, but there seems thus far to be no substantial encouragement that they are capable of accomplishing heavy work as economically as the higher grades of the reciprocating pumping-machinery. Even in moderately low-lift pumping a well-designed reciprocating pump will work more economically than the centrifugal pump, assuming either type of pump to be operated by an equally efficient steam-engine. The only element of annual expense which will weigh against the reciprocating pumping-engine and in favor of the centrifugal pumping-engine is the interest on the cost of manufacture, transportation, and installation. When this element of charge, taken in connection with the annual cost of operation and maintenance of the reciprocating type of machine, exceeds similar expenses in the aggregate of the centrifugal type of machine, the latter installation becomes the cheaper. When the lift is very low and very large volumes of water are to be handled periodically the centrifugal-pump installation becomes the more economical. The centrifugal pump as ordinarily manufactured is of comparatively low efficiency, and, so far as we may judge from the few reliable tests of such pumps, the efficiency is more often below than above 50 per cent, and sometimes as low as 35 per cent. Intelligent design and careful shop-work are necessary to produce an efficiency of 60 to 70 per cent.

The centrifugal or turbine pump gives its best efficiency when running at a constant speed and delivering water against a uniform head and is therefore poorly adapted to that condition of service which requires occasional change from a reservoir delivery to a direct-pressure delivery.

The rotary motion of this class of pumping-machinery peculiarly adapts it to a direct connection with an electric motor wherever occasion requires pumping by transmitted power.

A pumping-engine of any grade does its best work when moving constantly at its rated capacity and delivering water against a steady pressure, as when delivering water into a reservoir. But reservoir-pumping warrants the installation of the highest grade

of pumping-machinery only when the volume of water-consumption is sufficient to require the continuous use of pumps of large capacity. When large-capacity machinery is so operated under skilled mechanical supervision it should give the highest attainable station duty.

The same class of machinery operated under direct-pressure service gives a lower station duty because in a service of this kind the fluctuating rate of consumption produces a fluctuating rate of speed, and correspondingly a momentary fluctuation of duty which in the average is considerably below the duty of the machine moving uniformly at its best speed under steady reservoir pressure.

As a rule it does not pay to install high-duty pumping-engines in localities where the water-lift is light and the daily period of service short; besides the local cost of fuel is to a considerable extent an influencing factor. Small towns very properly install comparatively low-duty engines during the probationary period and gradually increase the duty of future installations as the consumption and hours of service progressively increase.

The life of a pumping-engine depends upon the carefulness and intelligence with which it is operated and kept in repair, quite as much as upon the design and construction for the particular work which it is required to perform in daily practice. The wear and tear, or in other words the cost of maintenance, depends very much upon the simplicity of design of the machine, perfection of workmanship, rigidity of imporant parts, and stability of installation, and the rate of deterioration of moving parts depends in a great measure upon the number of reversals in any unit of time. That is to say, high-speed operation generally contributes in a greater degree to the cost of maintenance than does low-speed operation.

The mechanical life of a machine which is under good attendance and is not overworked may be very long, but its actual life depends very much upon the rate at which it is called on to work. Whenever its capacity becomes exceeded or daily requirements of work reach a point where the machine of itself fails simply because of incapacity, its usefulness as an independent factor becomes impaired although mechanically it may be altogether reliable. This situation can be met by the installa-

tion of larger machinery, but it does not necessarily require the complete abandonment of the older machine, which may yet perform important and satisfactory service as a reserve machine.

Instances arise where the actual substitution of one machine for another in the daily service produces a saving in station expenses which warrants the change regardless of the fact that the older machine may be thoroughly reliable mechanically, but the act of substitution cannot be interpreted as reflecting complete mechanical deterioration of the abandoned machine, so long as it remains a part of the station equipment and can be used in a secondary if not in a primary capacity. The benefit of the change is to be found directly in the decreased station expenditure resulting from the use of the more economically working substitute for the heavy daily service. The complete abandonment of the older machine as a part of the station equipment will seem to be justified only when the extra expense of operating and maintaining it in a subordinate capacity, plus interest on the investment in its purchase and installation, is considerably greater than the sum of similar charges against an installation of a higher grade but more costly machine intended for work in a similar subordinate capacity.

The basis of a guaranteed duty for some specified set of conditions is the proper basis for the purchase of pumping-machinery. But the basis of purchase is seldom adopted in the purchase of the small direct-acting pumping units installed in small waterworks, for the reason that the volume of water pumped is so small and the daily period of service so short that the cost of the fuel consumed in actual pumping-service is small in comparison with the other annual station expense; frequently the saving in fuel resulting from the use of a more efficient type of pumping-machinery will not amount to the interest on the difference in cost of the two installations.

However, as the volume of water pumped increases and the daily period of service lengthens, the need of a higher grade of pumping-machinery becomes necessary and apparent, and naturally the duty basis of purchase becomes an independent and usually a controlling consideration.

The penalty for non-fulfillment of the duty guarantee sometimes takes the form of a provision retaining a percentage of the

purchase-price until a proper test of the pumping-machinery installed shall have demonstrated its ability and capacity to give the duty guaranteed of it; at other times a forfeit is demanded upon the basis of a specified sum to be forfeited for each one million foot-pounds of duty the work of the engine falls below the guaranteed duty. In this regard the manufacturer has good reason for insisting that a forfeiture provision of the latter character should be balanced by a premium clause offering some specified bonus for each one million foot-pounds of work the engine proves itself capable of performing in excess of the guaranteed duty. Unless there is some balancing provision of this character the forfeiture clause of itself may result in higher bids than would otherwise be the case.

Such provisions, however, are customary, as a rule, with large and high-class pumping-machinery only.

CHAPTER IV.

IMPOUNDED SUPPLIES.

THE flow of rivers and streams, the waters of which are of sufficient purity for use without purification, is usually less during certain periods of a year than the demands for water-supply purposes. To insure an adequate supply during periods of low stream-flow, the water of freshets and flows in excess of the consumption by the works is stored in impounding or storage reservoirs. These may be natural lakes and ponds, or artificial reservoirs formed by the construction of dams across the valleys through which the streams flow. In many instances artificial reservoirs are reproductions wholly or in part of ancient lakes.

The water from a storage reservoir is to be preferred to that taken directly from a stream, inasmuch as the quality of a surface-water is improved by storage in a reservoir of sufficient capacity which has been properly prepared for service. The effects of pollution by organic impurities are more marked in instances where water is taken directly from a stream than where the supply is obtained from a large storage reservoir, since the period which elapses from the time of contamination to that of consumption in the first instance is usually insufficient to result in the elimination of the germs of typhoid fever and other intestinal diseases. The fact that epidemics of typhoid fever have been due to polluted surface-water supplies taken directly from streams is at present well established, and no community supplied in this manner is safe from these epidemics when the catchment area of the stream is inhabited.

In all probability pathogenic organisms do not multiply in water-supplies, and experiments made by biologists indicate that

these organisms do not retain their vitality longer than one month in the water of a reservoir. Consequently storage reservoirs act as safeguards against epidemics of intestinal diseases.

The period of quiescence which the water of a large reservoir undergoes is beneficial, since the turbidity of the entering water during freshet flows in particular is considerably if not entirely reduced as a result of sedimentation. The color of a water is also reduced by storage in consequence of the bleaching action of sunlight. This effect is shown by the following table.*

TABLE SHOWING THE EFFECT OF LONG STORAGE UPON THE COLOR OF WATER.

Locality.	Percentage of watershed above inlet.	Average time required to fill the reservoir, months.	Color at main inlet.	Estimated average color of all water entering reservoir.	Color at outlet.	Change.
Boston,						
Reservoir 3....	79	1.3	1.00	0.86	0.82	−0 04
Reservoir 2....	97	0.4	1.10	1.07	0.98	−0.09
Lake Cochituate	40	8.5	0.86	0.58	0.25	−0.33
Reservoir 4.... (Ashland)	84	7.5	1.40	1.27	0.72	−0.55
Brockton Reservoir.....	82	3.4	2.24	1.87	0.89	−0.98

It should be noted that the color of a stored water is not materially reduced unless the period of storage is about eight months.

The beneficial effect of long storage upon the color of a water is, however, materially affected by the condition of the reservoir bottom. When the reservoir site has been carefully prepared by the removal of organic matter and the entering water contains little or no matter in suspension the best results are secured. On the contrary when the bottom of the reservoir is muddy the color of the bottom layers in deep reservoirs is increased during the stagnation periods which are common to reservoirs of a depth greater than twenty feet. The following tables illustrate the effect of organic deposits in a reservoir upon the color of the contained water.†

* Stearns and Drown. "Discussion of Special Topics relating to the Quality of Public Water-supplies." Report of Mass. State Board of Health on Water-supply and Sewerage, Vol. I. 1889.

† Boston Water Reports, 1892, pp. 96, 97.

COLOR OF WATER IN LAKE COCHITUATE AT DIFFERENT DEPTHS.

(Depth 60–65 feet.)

1890.	Surface.	Mid-depth.	Bottom.
August 5	0.10	0.20	0.80
" 11	0.10	0.20	2.80
" 19	0.15	0.30	3.80
" 26	0.15	0.30	2.80
September 3	0.10	0.30	2.30
" 9	0.10	0.30	2.60

COLOR OF WATER IN BASIN 4 (ASHLAND RESERVOIR) OF BOSTON WATER-WORKS AT DIFFERENT DEPTHS.

(Depth 30–35 feet.)

1890.	Surface.	Mid-depth.	Bottom.
August 5	0.50	0.50	0.50
" 13	0.35	0.50	0.60
" 18	0.40	0.50	0.80
" 26	0.50	0.45	0.80
September 3	0.40	0.40	0.70
" 9	0.40	0.45	0.70

The Ashland Reservoir was prepared by the removal of all the loam from the site, and at the time the samples were taken the bottom was composed of clean material.

During the circulation periods the water of the lower layers of deep lakes and reservoirs is mingled with the nearly colorless water near the surface, thus changing the color of the entire mass. Not only is the color of the water affected in the manner indicated, but the quality of a water obtained from a reservoir containing organic deposits in contact with the water is injured in other ways. The water of the bottom layers coming in contact with organic matter during the period of stagnation is deprived of its free oxygen and becomes charged with organic matter in various stages of decomposition. This foul water when mingled with the upper layers gives rise to disagreeable tastes and odors due to the matter in solution and suspension, or to the organisms which derive their food-material from this matter.

"The effect of the character of the reservoir upon the stored water is best indicated in a chemical analysis by the free ammonia, which is a product of decomposition, and the albuminoid ammonia, particularly the suspended portion of the latter, which indicates the abundance of organisms and other suspended organic matter

contained in the water." * Analyses of water from two cleaned and three uncleaned reservoirs in Massachusetts are given below.

Reservoir.	Condition.	Date of filling.	Date of collecting sample.	Number of months between dates.	Ammonia			
					Free.	Albuminoid.		
						Total.	Dissolved.	Suspended.
Boston, Reservoir No. 4 (Ashland)	Cleaned	April 1886	Aug. 1887	16	0.0005	0.0229	—	—
	"	"	Aug. 1888	28	0.0002	0.0286	0.0254	0.0032
Cambridge, Stony Brook Reservoir	"	Aug. 1887	Aug. 1888	12	0.0010	0.0288	0.0234	0.0054
Quincy Reservoir	Uncleaned	Dec. 1888	Aug. 1889	8	0.0200	0.0466	0.0386	0.0080
" "	"	"	Aug. 1890	20	0.0110	0.0360	0.0272	0.0088
Lynn, Glen Lewis Pd.	Uncleaned	Dec. 1889	Aug. 1890	8	0.0632	0.0902	0.0746	0.0156
Lynn, Walden Pond	"	Dec. 1889	Aug. 1890	8	0.0576	0.0746	0.0560	0.0186

The reduction of color due to long storage and the effect of cleaning a reservoir site is shown by Figs. 33 and 34. The changes during a year in the color of the influent streams, the surface-water, and the water at the bottom of two reservoirs in Massachusetts—a clean reservoir (Ashland) and an uncleaned reservoir (Lake Cochituate)—are indicated on these diagrams.

The micro-organisms producing disagreeable odors in water-supplies are not usually present in flowing streams, but find the most favorable conditions for their growth in the quiet waters of lakes and reservoirs. The extent of their development depends in large measure upon the available food-supply, and clean water stored in a clean reservoir presents the least favorable conditions for their existence. The nitrogenous matter contained in the water of the entering streams or derived from the bottom and sides of the reservoir itself affords means for their growth often in considerable numbers. Even if the reservoir site be carefully prepared, the water entering the reservoir from an inhabited catchment area, although perhaps thoroughly purified, usually contains nitrogenous material, and a storage reservoir is rarely free at all times from micro-organisms.

The experience in Massachusetts has been that trouble due to taste and odors of water-supplies is greatly reduced by the removal of material containing more than four per cent organic matter

* Report of Mass. State Board of Health, 1890, p. 33.

from the reservoir sites, or the covering with clean sand and gravel of deposits too deep to be economically removed. Shallow margins are avoided either by excavating material until a sufficient depth

FIG. 33.

is secured, or filling along the shores to make clean beaches. The excessive growth of grass and weeds along the shores during periods of low water is thus avoided.

In addition the quality of the water entering a prepared reservoir is improved by measures taken to drain swamps upon the catchment area, to provide for the interception of surface- and ground-water before it reaches these swamps and its conveyance to the reservoir, and for the disposal or treatment of sewage from buildings upon the area of catchment.

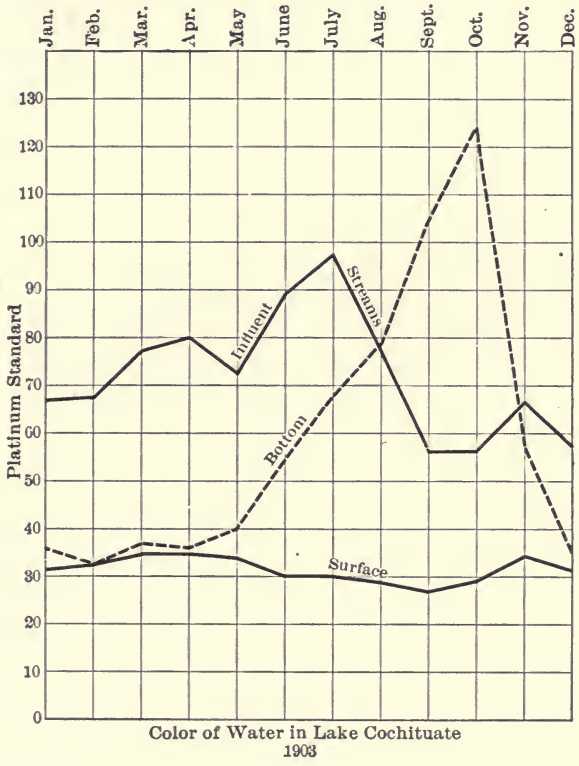


FIG. 34.

In the case of natural lakes or ponds which are subject to periods of stagnation and circulation, the effect of these occurrences upon the quality of the water is usually not so marked as has been indicated with respect to artificial reservoirs. Indeed, the view is often advanced that the stripping of a reservoir site is an item of unnecessary expense inasmuch as in course of time through the dissolving action of the water the bottom of a reservoir will approach a condition similar to that of a natural lake.

The length of time required to accomplish this end is, however, very uncertain, and unless the benefits of stripping are liable to be impaired by deposits washed into the reservoir by freshets it is considered advisable to expend funds for the purpose of removing objectionable material from the bottom and sides of an impounding reservoir.

An objection to a supply of water from a shallow impounding reservoir is based upon the comparatively high temperature of the water during the summer months. The temperature of the upper

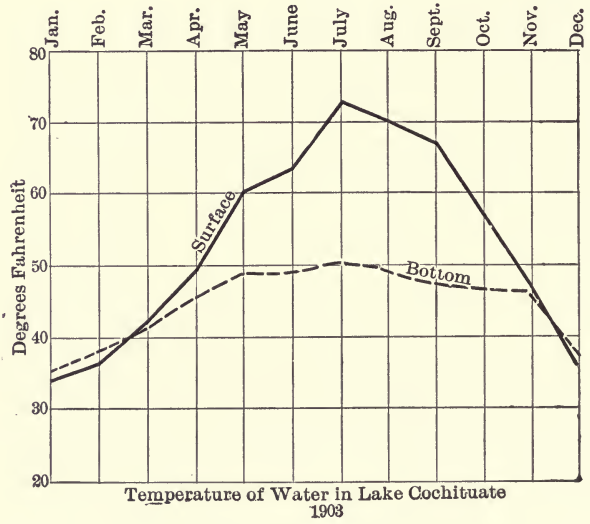


FIG. 35.

layers is favorable to the growths of objectionable organisms, and unless the reservoir is clean, water cannot be advantageously drawn from the cooler layers near the bottom even when the reservoir is deep. The warm water from a shallow reservoir is not looked upon with favor by consumers, as the water, even if unaffected by organisms producing tastes and odors, is not very palatable unless cooled. Owing to the evaporation losses from large areas of water-surface and the high temperature of the water of shallow reservoirs during warm weather, impounding reservoirs should be constructed of considerable depth.

The capacity of an impounding reservoir in any given instance

is dependent upon local conditions, the amount of storage to be provided being computed from actual or estimated figures for yield and consumption. The yield used in these computations should not be the average yield of the catchment area under consideration, but the yield during a dry year or a succession of dry years. In the absence of definite information regarding the rainfall on or run-off from a catchment area, comparisons are made with areas for which suitable records are available. In making these comparisons, allowances are made for differences due to topographical and geological conditions affecting the yield of the respective areas. Such conditions are the relative altitude of the areas, distance from the sea or large inland lakes, and the proximity of mountain ranges, etc., as affecting the rainfall; the size of the catchment area, character of vegetation upon the area, extent of forests, general slope of surface, amount of impervious surface, character of the material composing the different strata, temperature of the air, etc., as affecting yield. Furthermore, the losses by evaporation from the water surfaces in the area, and the losses due to percolation or seepage from the reservoir and the area itself are considered. Tables or diagrams are prepared giving the actual or estimated rainfall, run-off, consumption, evaporation, and other losses by months from which the amount of storage required is computed.

The danger of using average values for rainfall and yield in storage computations is shown by the table of statistics for the Sudbury catchment area. The average results obtained from 1875 to 1904, figures for the two years in which the yield was the lowest recorded and figures for the year in which the yield for six months was a minimum, are given in this table. The fluctuations during the period covered by the observations are shown in Fig. 36.

The matter of storage on the Sudbury area has been carefully studied by the engineers connected with the Boston works, and the results of these studies are contained in Mr. Fitzgerald's paper on "Rainfall, Flow of Streams, and Storage." * The diagram of storage capacity is prepared from that given in the paper mentioned and is applicable to catchment areas in New England comparable

* Trans. Am. Soc. C. E., Vol. XXVII.

Yield of Sudbury Catchment Area.

	Average 1875–1904.			1880.			1883.			1899.		
	Rainfall. Inches.	Collected. Inches.	Yield gallons per day per square mile.	Rainfall. Inches.	Collected. Inches.	Yield gallons per day per square mile.	Rainfall. Inches.	Collected. Inches.	Yield gallons per day per square mile.	Rainfall. Inches.	Collected. Inches.	Yield gallons per day per square mile.
January...	4.21	2.17	1217000	3.57	2.000	1120000	2.81	0.597	335000	4.18	4.082	2288000
February..	4.34	3.02	1861000	3.98	2.982	1787000	3.87	1.664	1033000	4.91	2.225	1381000
March.....	4.59	5.33	2986000	3.31	2.451	1374000	1.78	2.873	1611000	7.01	7.501	4205000
April.....	3.61	3.65	2116000	3.11	2.017	1169000	1.84	2.330	1350000	1.90	4.351	2521000
May.....	3.34	2.04	1141000	1.84	0.917	514000	4.19	1.673	937000	1.45	0.911	511000
June.....	3.09	0.90	522000	2.14	0.303	175000	2.40	0.518	300000	2.51	0.114	66000
July.....	3.67	0.35	193000	6.27	0.315	176000	2.68	0.206	115000	3.22	0.035	19000
August....	4.05	0.52	291000	4.01	0.212	119000	0.73	0.140	79000	1.43	−0.063	−35000
September.	3.32	0.41	238000	1.60	0.138	80000	1.52	0.157	91000	3.95	0.162	94000
October...	4.23	0.90	506000	3.74	0.181	102000	5.60	0.331	186000	2.69	0.206	115000
November.	3.96	1.50	868000	1.78	0.354	205000	1.81	0.354	205000	2.18	0.525	304000
December.	3.81	1.93	1082000	2.83	0.312	175000	3.55	0.345	194000	1.78	0.392	220000
Total	46.22	22.72		38.18	12.182		32.78	11.188		37.21	20.441	
Average for year			1081000			578000			533000			973000
Average for driest six months			435000			143000			145000			93000

to the Sudbury. Results obtained by the use of this diagram are modified by the statement that "as it is undesirable to keep the water below high water for more than two years in suc-

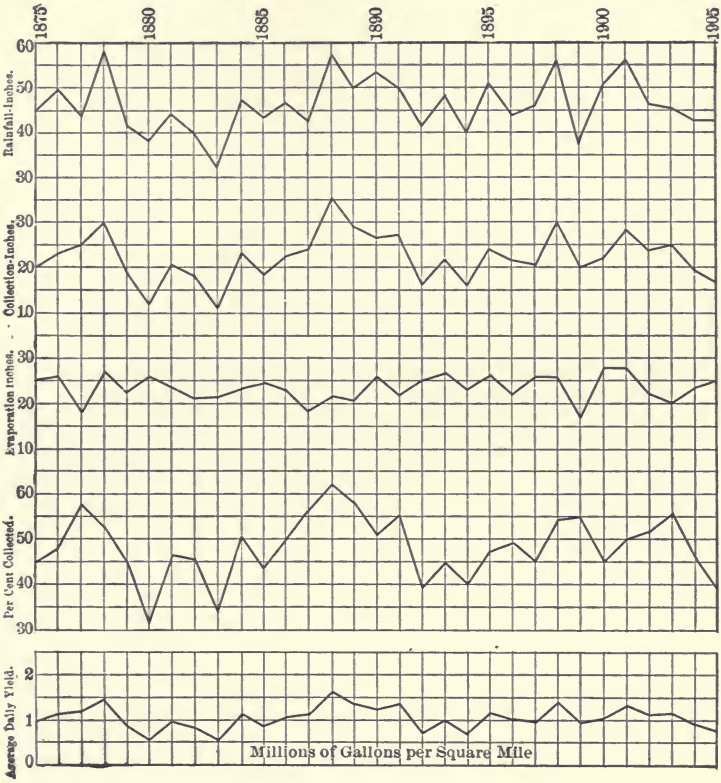


FIG. 36.—Diagram of Yield of the Sudbury River Catchment Area.

Catchment area: 1875–1878 = 77.764 square miles
1879–1880 = 78.238 " "
1881–1905 = 75.2 " "
Per cent of water-surface: 1875–1878 = 1.9%
Increased in 1879 to 3.0%
" " 1885 " 3.4%
" " 1894 " 3.9%
" " 1898 " 6.5%

cession, . . . it is impracticable to secure more than about 750,000 gallons daily from 1 square mile of watershed containing 10 per cent of water-surface."* To secure this supply a storage

* Trans. Am. Soc. C. E., Vol. XXVII, p. 268. ("Rainfall, Flow of Streams, and Storage." Fitzgerald.)

capacity of about 222 million gallons per square mile is required. This is equivalent to about 56 per cent of the average annual yield.

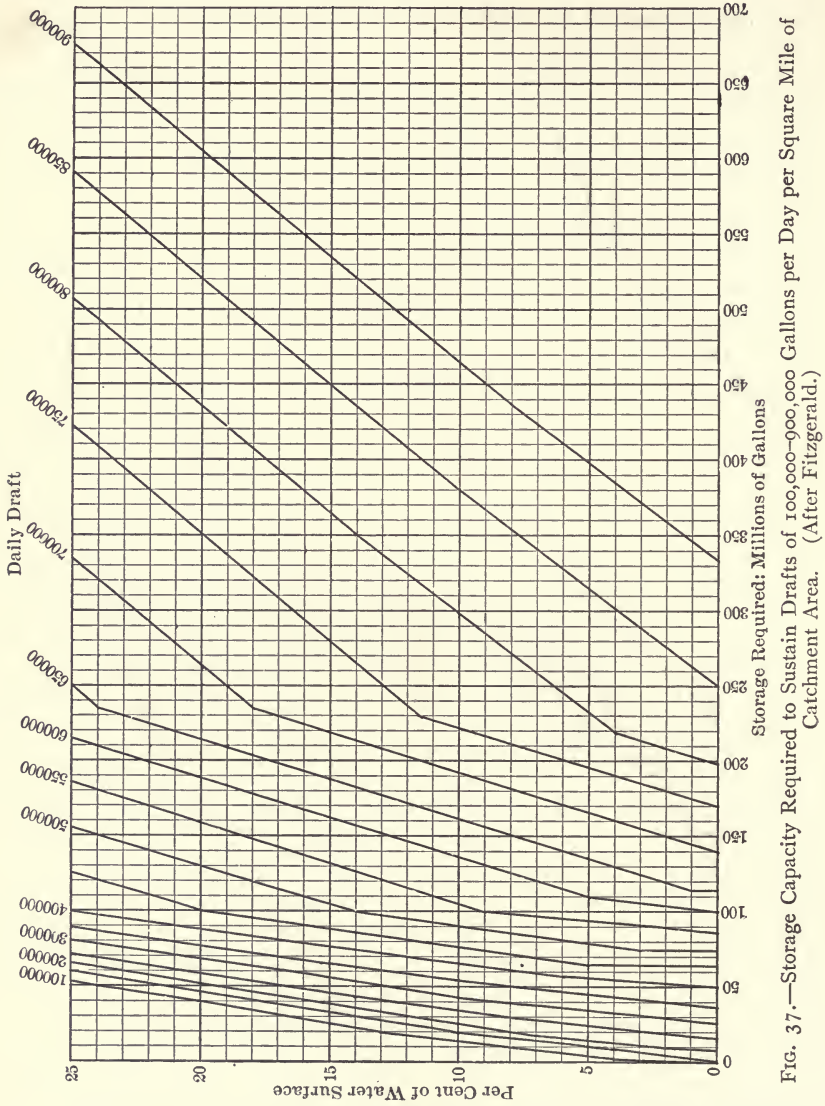
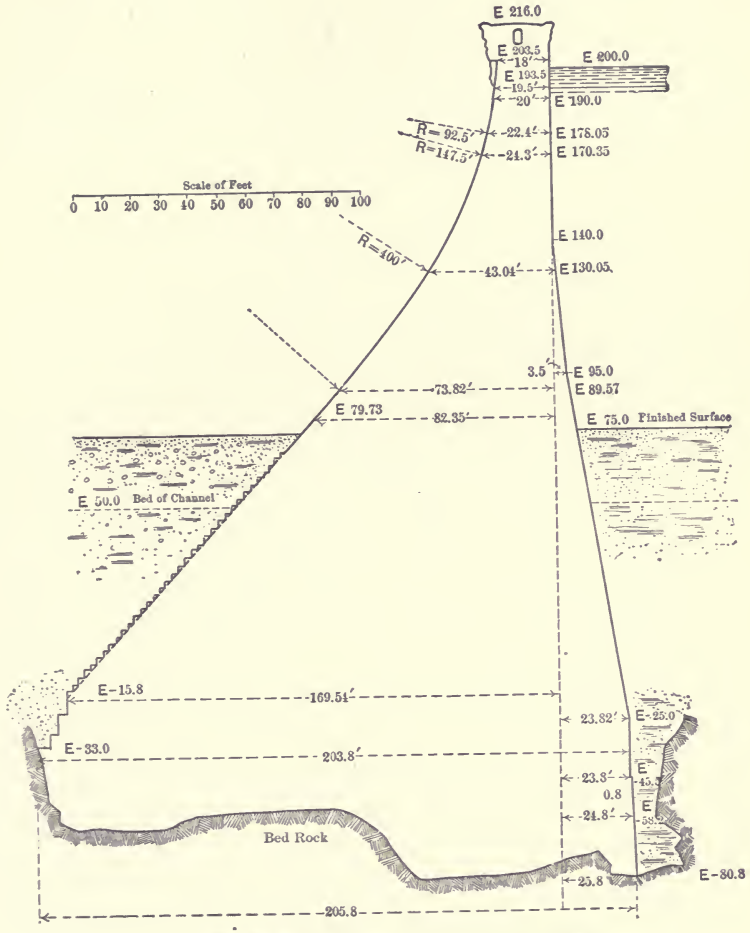


FIG. 37.—Storage Capacity Required to Sustain Drafts of 100,000–900,000 Gallons per Day per Square Mile of Catchment Area. (After Fitzgerald.)

The effect of varying percentages of water-surface upon the area of catchment is clearly shown by the diagram, and the extent of water-surface should be kept below 10 per cent of this area whenever practicable. In the case of natural lakes and ponds in



MAXIMUM SECTION OF NEW CROTON DAM.

FIG. 38.

New England where the proportion of water-surface is comparatively large, the available storage may be from one to one and one-half times the average annual yield.

Dams or embankments are a feature of all artificial reservoirs, and often the selection of a reservoir site depends upon the practicability and cost of the necessary structures to retain the water therein. Provision is also made for the disposal of the water from freshets which cannot at all times be stored. Unless the main dam is designed to pass water, which is not usually the case with any but very low structures, masonry spillways or waste-weirs should be constructed of sufficient capacity to waste the water from the maximum known, or anticipated, flood without danger of overtopping the dam. The site of the spillway depends upon the topography of the valley in which the reservoir is situated and the proximity of the bed-rock to the surface. When the dam is of considerable height the spillway is located at one side of the valley if possible, with its crest from six to twenty feet below the top of the main structure.

Dams of a height exceeding about seventy-five feet, when built in connection with reservoirs of considerable extent, are usually constructed of masonry. The masonry may be rubble, concrete, or cyclopean faced with ashlar or concrete blocks to make a neat finish. The foundations of high masonry dams are carried to sound bed-rock. Fig. 38 shows a typical cross-section of a modern high masonry dam, the New Croton, which is the highest yet constructed.

Rock-fill dams have been used in some instances in the West where cement could not be economically procured. Dams of this class are made more or less water-tight by a facing or core of concrete, timber, or protected steel plates.

Earth is largely used in the construction of dams and embankments, gravelly earth being considered the best material for this purpose. Earth dams are, or should be, provided with a core-wall of masonry unless the bank is of unusual thickness and in no danger of ever being overtopped. The core-wall is usually of concrete and is carried to bed-rock on the bottom and sides of the valley when this can be done with economy. Otherwise the core-wall is founded on hard-pan or other impervious material. Sheet-piling has been

used in some instances beneath the core-wall when the rock was at a considerable depth.

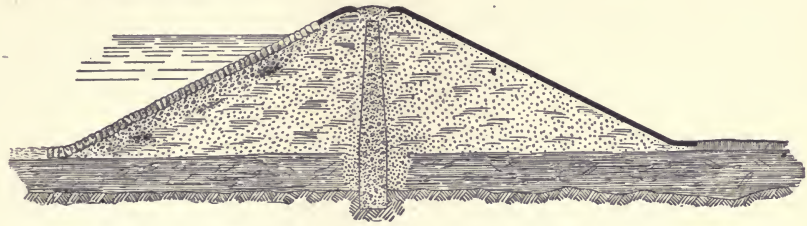


FIG. 39.—Cross-section of Earth Dam.

Earth dams of ordinary height vary in thickness from 12 to 30 feet at the top and have side slopes from $1\frac{1}{2}:1$ to $3:1$. The top is usually from 6 to 10 feet above high-water level. The slopes on the water side are paved with stone to the height reached by waves, and the down-stream slopes are sodded or faced with loam and sown to grass.

When the height of the embankment exceeds 30 to 40 feet a berm is generally built at about mid-height on the water side to guard against the slipping of the stone paving. A similar berm is also placed on the down-stream side to prevent washing of the material by rains. Below the berm on the water side the slope is often riprapped with heavy stones as a substitute for the paving.

The outlet-pipes from the reservoir and the controlling-valves are placed in gate-towers or chambers which are usually built in connection with the masonry portion of the dam or, in the case of earth embankments, as independent structures.

PART II.

MAINTENANCE AND OPERATION.

CHAPTER I.

PLANS AND RECORDS.

An important requisite for the successful management of a system of water-works is a complete set of plans and records. These should be accurate, should be kept up to date, and should admit of ready interpretation. Dependence should not be placed upon scattered and fragmentary notes in numerous field and office note-books, nor upon the memory of any person or persons having charge of, or connected with, any portion of the works. The value of such plans and records may not be apparent at the outset, but the changes which inevitably take place through the growth of a community, with the resulting obliteration of old landmarks, the relocation of street-lines, the changes in the personnel of the department, and so on, very soon indicate that funds expended for the proper preservation of data have been expended wisely.

Construction Plans.

In the case of surface-water supplies, maps or plans should be prepared, upon which are indicated the source of supply, the catchment area, all water-surfaces, buildings, dams, and intakes. These plans or notes thereon should furnish information regarding

the size of the catchment area, and the extent of water-surface at varying elevations for use in making computations of yield.

Detailed plans of all pipes and appurtenances in, and in the vicinity of, gate-chambers, pumping-stations, filtration-plants, reservoirs, and stand-pipes are to be desired.

Accurate plans of the distribution system should be prepared from surveys made, or based upon measurements taken, during the work of construction. These plans may vary in character from the carefully drawn atlas sheets on a scale of forty or fifty feet to the inch, upon which are indicated the main pipes, valves, hydrants, plugged branches, service connections, street- and curb-lines, and all buildings, to the single sheet upon which main pipes, valves, hydrants, and street-lines alone are shown on a scale which will admit of the plotting of the distribution system upon a single plan, as the funds available for this purpose may determine.

The record plans should preferably be made upon heavy paper mounted on linen, as plans on tracing-cloth are not very durable and in addition cannot be accurately scaled owing to the shrinkage of the material. Copies of the plans in the form of tracings or blue-prints made therefrom may, however, be of service. The several sizes of pipe may be indicated upon the plans by inks of different colors, valves by short heavy black lines at right angles to the pipe-line, and hydrants by small circles. In suburban localities where street-lines are not marked by monuments, or are otherwise not plainly defined, the travelled way may be shown to advantage by light dotted lines.

A skeleton plan of the entire distribution system, either drawn to scale or carefully sketched, with the main valves indicated will be found useful for many purposes, especially in connection with a valve-location book.

In order that the valves on street-mains may be quickly located in time of need, a valve-location book is indispensable. Where valves are set on the street-lines, tie locations may not be necessary when the distance of the valve from the line of the side street is known and both street-lines well defined, but usually locations by ties to permanent objects are advisable. A sketch of the immediate vicinity of the valve should be made, and measurements taken to the box-cover from the corners of adjacent build-

ings, trees, hydrants, light-poles, etc. At least three ties should be taken from objects, as noted above, which are not likely to be covered by snow, the distances measured being preferably such as may be made within the limits of a fifty-foot tape. Copies of the sketches should be made upon tracing-cloth and the measurements with the points from which they are to be taken clearly indicated. Blue-print sets of valve locations can then be bound in book form for convenient use. The size of the sheets should admit of the carrying of the book in the pocket, sheets four by

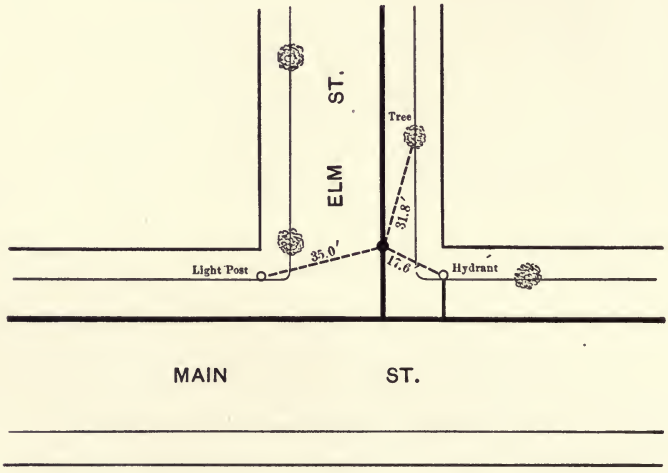


FIG. 40.—Sketch of Location of Main Valve.

six inches being suitable for the purpose. In cities a departure from the method of measurements outlined may be made by marking points at equal distances from the valve-box by bolts in, or circles marked in paint on, adjacent buildings.

Similar tie-locations may be made of plugged branches with the addition of notes indicating the distance of the branch from the nearest valve or hydrant branch.

Since service-pipes as a general rule are laid at right angles to the axis of the street, the curb-cock alone may be located by measurements from the corners of the building to which the pipe is laid or from other convenient objects. The diagrams of the services and the ties to the service-boxes may be made upon cards

which are filed alphabetically, according to the name of the property owner or occupant of the premises, or by streets and numbers, at the option of the official in charge of the work. The corporation cock or tap may be located by similar measurements if desired, or both curb and corporation cocks may, in the case of isolated buildings, be located by distances along, and offsets from, the line of the building produced. This latter method of location, however, possesses no advantages over that first mentioned, and

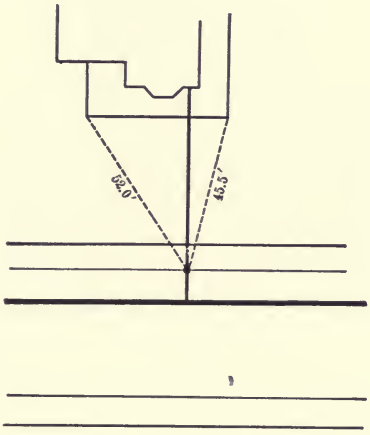


FIG. 41.—Diagram of Service Connection.

relocations cannot be made as accurately as by the use of measurements which intersect at a point.

Construction Records.

The plans of a system should be supplemented by records kept in specially designed books, on sheets preserved in loose-leaf binders, or by the card system. The first method is preferable for the preservation of data regarding street mains, valves and hydrants, and the second or third in the case of service connections, particularly when the service application and data are entered on a single form. In addition to the statistics of original construction, information respecting alterations, renewals, and discontinuances should be recorded.

The record-books may be arranged in such manner that information concerning the separate items may be placed on a horizontal line; the data being entered under appropriate headings printed above vertical columns covering two opposite pages of the book, or one page if an oblong book is used. The following headings are suggested for main-pipe, valve, and hydrant records:

Main Pipes: Street; Location; Length and size of main (separate columns being used for the several diameters of pipe used); Distance from street-line; Material; Number of valves; Number of hydrants; Length and size of hydrant branches; Length and size of blow-off branches; Cost—pipe, specials, valves, valve-boxes, hydrants, lead and yarn, labor, teaming; Date of completion, etc.

Valves: Street, Location; Size (separate columns being used for the several sizes); Type of valve; Maker; Turn to open, etc.

Hydrants: Street; Location; Size of main; Length and size of branch; Number and kind of nozzles; Type; Maker; Turn to open, etc.

The service records may be entered on printed sheets or cards, the service application and data appearing on one side of the form, the reverse being used for the diagram. These records should furnish information upon the following points: Owner and Occupant of Premises; Street and Number; Material of service-pipe; Diameter; Length—main to curb-cock, curb-cock to street-line, street-line to stop and waste; Total length; Number and size of taps; Date of completion; Cost—corporation-cock, stop-cocks, service-box, pipe, labor, teaming, etc. The items of cost may be divided in order that the respective costs within the limits of the street and in private property may be ascertained in cases where bills are rendered the property owner based upon actual expenditures for construction outside the limits of the street.

A specimen service application and record is shown on the following page. The sheets are $8 \times 10\frac{1}{2}$ inches in size and are filed in loose-leaf binders.

TOWN OF CONCORD: WATER DEPARTMENT.

Service Application.

SERVICE NO.....

CONCORD, MASS.....

To the Water and Sewer Commissioners:

The undersigned hereby applies for a supply of water for the premises on.....Street, owned by..... and occupied by.....as a.....and .....hereby agree to conform to all rules and regulations now or to be established by the Commissioners.

.....Applicant

RECORD OF SERVICE.

COST WITHIN STREET.		COST WITHIN PRIVATE PROPERTY.	
Foreman	$.....	Foreman	$.....
Labor	.....	Labor	.....
Team	.....	Team	.....
....ft....in.....pipe @	.....	....ft....in....pipe @	.....
....Stop and Waste Cock	.....	....Stop and Waste Cock	.....
....Corporation Cock	.....		.....
....Service Box	.....		.....
.....		Supervision and Use of Tools	.....
Total	.....	Total	$.....

DIAGRAM OF SERVICE.

Kind of Pipe	.....
Size of Pipe	.....
Main to Stop	.....
Stop to Line	.....
Line to Stop and Waste	.....
Total Length	.....
Number of Taps	.....
Service Completed	.....
Bill Rendered	.....
Bill Paid	.....
Remarks	

Maintenance Records.

Whenever possible, data with regard to the yield of catchment areas from which surface-water supplies are obtained should be systematically collected and recorded. Observations of the precipitation at one or more points on the area, of the elevations of the water-surfaces of ponds, lakes, or impounding reservoirs on the area of catchment, of the quantities of water drawn from the

area for use in the works, and of the amount flowing off in streams are necessary for this purpose.

The measurements of precipitation should be made with a standard gage. The instruments adopted by the U. S. Weather Bureau are described and directions for their use given in circulars published by that Bureau. (Measurement of Precipitation, U. S. Dept. of Agriculture, Weather Bureau, 1903; Instructions to Observers, U. S. Dept. of Agriculture, Weather Bureau, 1903.)

The standard gage of the Weather Bureau is shown by the accompanying illustration. This gage consists of a receiver (*A*) which is 8 inches in diameter inside, a measuring-tube (*C*) of such

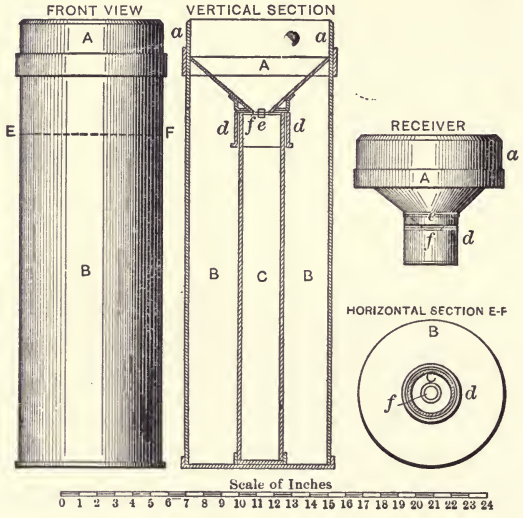


FIG. 42.—Standard Rain Gage.

diameter that the depth of rainfall collected is magnified ten times, a graduated measuring-stick and an overflow attachment (*B*), in which rainfall in excess of the capacity of the measuring-tube may be collected. Rainfall is measured in inches and decimals of an inch, and precipitation in the form of snow is recorded in equivalent inches of rainfall. Snowfall is collected in the overflow attachment of the gage and carefully melted. Observations of precipitation should be made daily. The gages are preferably exposed in an open space and at a distance from trees, buildings, or high fences. "Low bushes and fences, or walls that break the

force of the wind, are, however, beneficial, if at a distance not less than the height of the object." The top of the gage is placed level and about three feet above the ground.

The elevation of the water-surface of ponds, lakes, or storage reservoirs should be observed periodically, and the increase or decrease in the amount of water in storage computed from the results of these observations and the data regarding the extent of water-surface at different elevations. Tables or diagrams that indicate the volume of storage corresponding to stated elevations of the water-surface are useful in this connection.

The draft from the catchment area is ascertained from pumping or meter records, or observations of the flow of water over weirs. The first-mentioned records are likely to be seriously in error, unless the slip of the pumps be known and provided for in the computations of pumpage.

The ordinary type of water-meter is not, as a rule, used for the measurement of the total volume of water supplied to a city or town; the Venturi meter, an instrument devised by Clemens Herschel, C.E., being used for this purpose. The Venturi meter consists of a contracted tube composed of two truncated cones and a throat-piece which connects the smaller ends of the former. This tube is placed in the pipe-line. When water is flowing through the tube, the velocity of flow is greater at the throat than at the up-stream end of the tube, and the pressure at this point is less than that at the latter; the difference in pressure being dependent upon the quantity of water flowing. The loss of head due to the contraction of area of the pipe is, under ordinary circumstances, insignificant. The up-stream end and the throat of the meter-tube are connected by pipes with a recording-apparatus, which mechanically integrates the varying relations of velocity and pressure within the tube, and indicates on a dial the amount of water which has passed through the meter. By the addition to the register of a chart-recorder the rate of flow at intervals throughout the day is recorded upon a strip of paper. The common sizes of meter are from six to sixty-inch, and the meters are designed to measure flows when the maximum rate does not exceed about ten times the minimum rate of flow through the supply-main in which the meter is placed. The results obtained

by Venturi meters operating under suitable conditions are accurate within about two or three per cent. Forty-nine Venturi meters were used in investigations made by the Metropolitan Water and Sewerage Board in 1903; the arrangement of the meter and registering-apparatus as generally used being shown in Fig. 43.

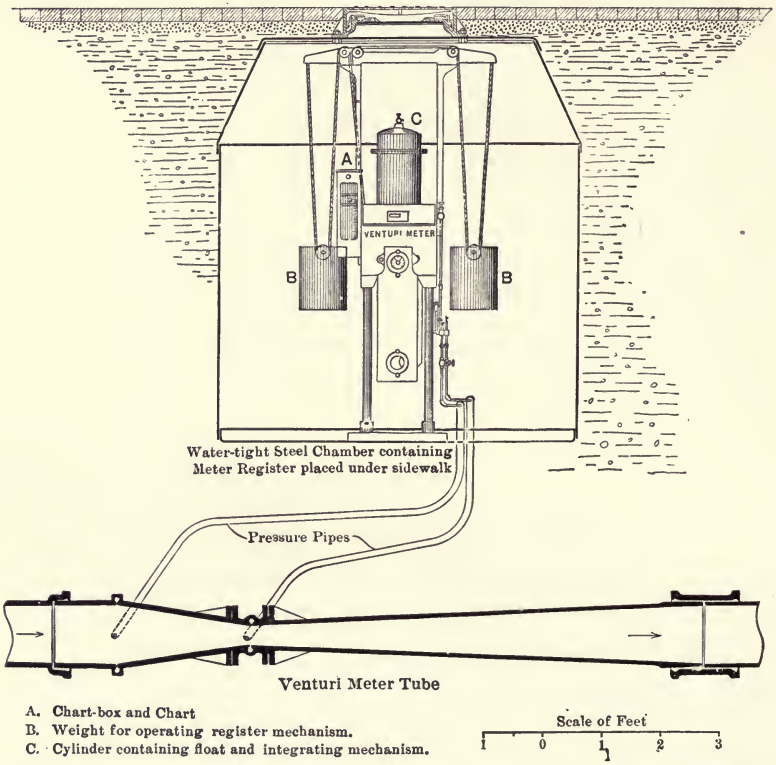


FIG. 43.—Venturi Meter and Register Chamber. (Jour. N. E. Water-works Assoc'n, Vol. XVIII, p. 113.)

The flow of water over weirs or dams is computed from observations made of the height of water above the crest of the weir or dam, by the aid of formulas adapted for use with the conditions which obtain in a given instance.

The yield of or run-off from a catchment area is dependent upon the precipitation upon the area, and the evaporation from the land and water surfaces. The amount of water available from the area is determined from the computations of yield or

run-off; these computations being based upon the observations of the quantity of water in storage, the draft, and the waste over dams or waste-weirs. This amount varies from year to year, and since the capacity of the source is dependent upon the minimum yield and the opportunities for storage, records extending over a period of years and embracing varied conditions are of great value when observations are accurately made and the data carefully compiled.

The average results obtained from observations made on some existing works are of interest in this connection and are given in the following table.

AVERAGE YIELD OF CATCHMENT AREAS.

Stream.	Period covered by observations.	Catchment area. Square miles.	Average Rainfall. Inches.	Average run-off. Inches.
Sudbury.....	1875–1904	75.2	46.22	22.717
Wachusett.....	1897–1904	119.0	50.07	26.742
Croton.....	1868–1899	338.8	48.07	22.93
Perkiomen.....	1883–1905	152.0	47.63	23.04
Neshaminy.....	1883–1905	139.3	47.82	22.85
Tohickon.....	1883–1905	102.2	48.91	27.47

The extent of water-surface on a catchment area affects the total evaporation from that area since the evaporation from a water-surface exceeds that from one composed entirely of land. In order that the yield of a catchment area may be computed on a net land basis, the loss from the water-surface must be known or estimated. Such computations are of value in the consideration of problems of additional supply, when the proposed sources are in close proximity to sources of which the yield is known, or in the consideration of the effect upon the yield of a source due to the construction of additional impounding reservoirs.

Measurements of the evaporation from a water-surface are made with a floating tub or tank of wood, protected in such manner that water may not be introduced into it from the pond or reservoir by the action of wind or waves. Results obtained on the Boston Water-works by Desmond Fitzgerald, C.E., from 1875 to 1890 indicated that the average evaporation from a water-surface in that vicinity was about 39.2 inches.*

* Trans. Am. Soc. C. E. "Rainfall, Flow of Streams, and Storage," Vol. XXVII; "Evaporation," Vol. XV.

This was distributed throughout the year as follows:

Month.	Evaporation in Inches.
January	0.96
February	1.05
March	1.70
April	2.97
May	4.46
June	5.54
July	5.98
August	5.50
September	4.12
October	3.16
November	2.25
December	1.51
Total	39.20

The average results obtained at the Mount Hope reservoir of the Rochester Water-works since the beginning of experiments in 1891 are as follows:*

Month.	Evaporation in Inches.
1896–1904 January	0.66
1896–1904 February	0.84
1896–1904 March	1.48
1894–1904 April	2.71
1892–1904 May	4.03
1892–1904 June	4.80
1891–1904 July	5.33
1891–1904 August	4.92
1891–1904 September	4.01
1891–1904 October	2.88
1895–1904 November	1.52
1895–1904 December	1.17
Total	34.35

The records of consumption are based upon the records of draft previously mentioned, with corrections applied to compensate for the gain or loss in the quantity of water contained

* Annual Reports, Rochester, N. Y., 1904.

in distribution-reservoirs or stand-pipes. Data with regard to the average, minimum, and maximum quantities of water consumed daily, weekly, or monthly are kept by a number of water-works departments. The maximum hourly rates of consumption from a system are of value in the consideration of problems of fire-protection.

Several recording devices are in use for the determination of the various elements entering into computations of yield, draft, or consumption. Continuous records of the height of water on dams or weirs, or in ponds, reservoirs, or stand-pipes may be

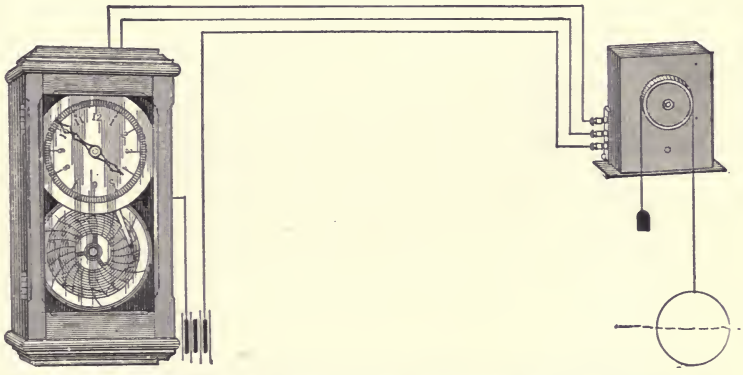


FIG. 44.—No. 2 Winslow Electrical Indicating and Recording Apparatus.

obtained by the use of apparatus consisting of floats, indicating devices, and rolls carrying record-sheets which are moved by clockwork. The apparatus illustrated is used for indicating and recording changes in water-level which take place at a point distant from that at which the recording instrument is placed.

When reservoirs or stand-pipes are filled by pumps, a device indicating the height of water in the reservoir or stand-pipe is usually placed at the pumping-station. An alarm-bell is often provided in connection with these devices which gives warning when the water-level approaches high-water mark. In cold climates the formation of ice on the reservoir or in the stand-pipe often limits the use of appliances depending upon a float for their operation, and pressure-gages are used for a like purpose, although the results obtained with these instruments are not as accurate.

Pressure-recording gages are placed in offices, pumping-sta-

tions, or workshops, and these instruments are often of value for indicating reductions in pressure due to large leaks or other causes, and as a check upon the operation of the pumps. Such gages should be attached to an independent supply-pipe leading from a large street-main, and the connection should not be made near branch-lines serving street-sprinkler stand-pipes or large consumers.

The engineer in charge of the pumping-station should be provided with blank forms upon which details with regard to the

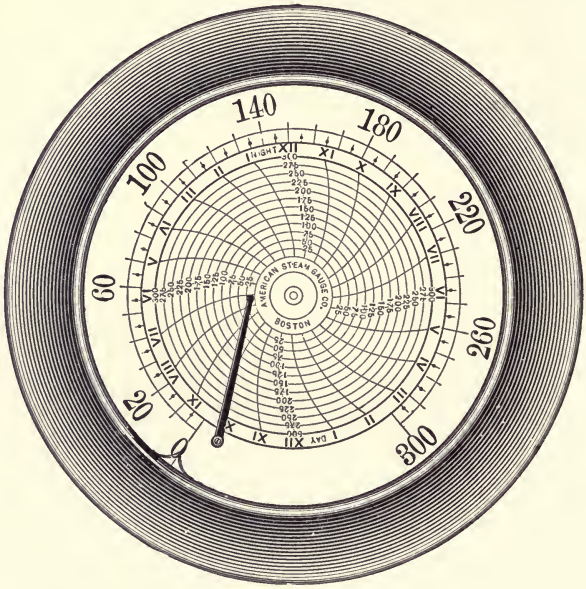


FIG. 45.—Combined Pressure and Recording Gage.

operation of the station should be noted daily. The pumping records should be kept in book form at the water-works office and the daily observations, with the computations based thereon, entered each month. The data may be recorded somewhat as follows: Date; time of starting pumps; time of stopping pumps; duration of pumping; reading of counter; number of strokes or revolutions; gallons pumped (with or without allowance for slip); reading of suction-gage; reading of pressure-gage; total lift (static and dynamic); reading of steam-gage; amount of coal

consumed; amount of ashes and clinkers; per cent of ashes and clinkers; quantity of feed-water; temperature of feed-water; duty. Care should be taken that the indications of the gages, scales, and water-meters are substantially correct.

When water is furnished at schedule or fixed rates, all premises and buildings supplied from the works should be systematically inspected at intervals and the results of the inspections entered in an inspection record. Dependence should not be placed entirely upon returns made by consumers or plumbers employed by them, as almost invariably fixtures are placed and, through the carelessness or negligence of the parties responsible for the work, no returns made to the water-works office. The items may be entered in the inspection record under the following headings: Service number; Street and number; Owner and occupant of premises; Number and kind of fixtures (separate columns are allowed for each class of fixtures ordinarily supplied and for which a charge is made); Use to which water is applied; Person to whom bill is made; Amount of bill; Remarks.

The water-bills are made from the data contained in this record.

Data with regard to meters purchased and placed by the water department should be preserved upon forms prepared for that purpose. A page in a meter-record book or a card, if the card system is in use, should be allowed for each meter and the following items entered thereon. Meter record: Size; Type; Make; Manufacturer's number; Date purchased; Cost; Date tested; Result of test; Date re-tested; Result of test; Quantity registered since previous test; Date set; Location; Date removed; Date reset; Location; Date repaired; Repairs necessary; Cost of repairs.

Records of the amount of water registered by service-meters should be kept in convenient form and the meter-bills made therefrom. The entries may be made in form somewhat similar to the inspection-record form previously mentioned except that in place of the items under the heading "Fixtures supplied" the number, size, and make of meter, the meter readings, and the quantities of water consumed are entered.

CHAPTER II.

EXTENSIONS.

EXTENSIONS of existing works are called for from time to time to provide for the constantly increasing demands of a community, and these additions to a system often require careful study. In some instances the extensions are made in accordance with a prearranged scheme evolved by the engineer responsible for the original design; in others the matter is left to the discretion of the board or commission in charge of the works. In the latter case it not infrequently happens that little or no regard is given to the possible future demands nor to the requirements for fire protection, with the result that many distribution mains soon prove inadequate, particularly for the latter purpose. It may be stated that as a general rule it is not advisable to make additions without recourse to the advice of a competent engineer.

Usually extensions of considerable extent may be made more economically through the medium of a good contractor, but small additions may be made by day labor. Before the work of construction is commenced, the location of the pipe-line should be marked on the ground and the position of specials, valves, and hydrants indicated by suitably marked stakes or by other means. A detailed plan or sketch of the proposed work should be placed in the hands of the foreman in charge, and no deviation from this plan should be permitted without the consent of the supervising engineer. In case surveys are not made during the course of construction, measurements should be taken to all points controlling the alignment, in order that such points may be accurately located on the surface when the surveys are carried out.

The main pipe-line should be laid at a uniform distance from the street-line, a distance approximately equal to one-third the

street width serving as a guide except at sharp curves, where, owing to the decreased friction losses, it is often advisable to use special fittings instead of making long bends in the alignment. The depth at which the main should be laid is dependent upon the locality and in cities, upon existing underground structures. The usual practice in New England is to excavate ordinary trenches from four and one-half to six feet in depth. However, since the frictional losses are increased by abrupt changes in the alignment in the vertical plane as well as in the horizontal, due regard should be given to the grade of the pipe-line.

Valves should be placed to control districts and streets in order that the water may be shut off for repairs. Valves located on branch lines should be set exactly on the line of the street in which the principal main is laid, even if such location necessitates the cutting of a pipe to accomplish this end. Hydrant connections from supply or other important mains should be provided with valves for use in making repairs to the hydrants.

In thickly settled communities hydrants should not be placed more than 500 feet apart, and in sparsely settled districts these should be located with a view to the ultimate location of hydrants at intermediate points. Plugged branches may be left at such points for future use. Since good drainage is essential to the proper operation of a fire-hydrant, care should be taken in their setting. In fine, compact, or clayey material the hydrant should be underlaid and the base surrounded with coarse gravel, bricks, or small field stones. The hydrants should also be backed with heavy stones, brickwork, or concrete, as instances are on record where a hydrant has been forced from the branch, with consequent damage to the street and adjacent property. Inasmuch as there is little or no circulation in the connections, care should be taken to set the inlet-valve and base of a hydrant below the depth ordinarily reached by frost. In order that the projecting nozzles may not be struck and broken by teams, hydrants should be set a short distance inside the curb-line.

Care should be taken in the work of construction to reduce the obstruction to traffic to a minimum, and all trenches, tool-boxes, etc., should be well lighted at night. For economy in operation the construction force should be divided into gangs, each performing

a special duty; for instance, one party may excavate the trench in advance, another attend to the digging of bell-holes and the laying of the pipe, and another the back-filling. Whenever possible, however, the pipe-line should not be covered until the water is turned on or the joints tested under pressure, although in order to prevent the caving of the banks the trench may be partially refilled between joints.

Long lines of pipe should not be left entirely exposed to the rays of the sun after the joints have been calked, as the expansion

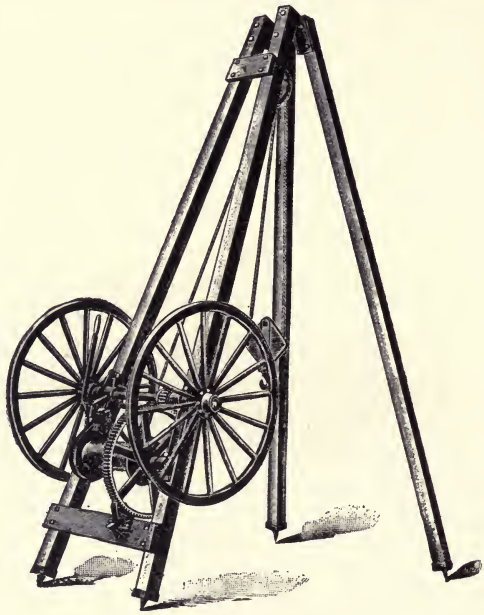


FIG. 46.—Pipe-derrick.

and contraction of the line resulting from the heating and subsequent cooling tends to start the joints.

Stones, sticks, or other material should be removed from the pipes before they are lowered into the trench, a brush attached to a long pole being of service for cleaning the smaller sizes, and the end of the pipe-line should be protected at night or at cessation of work by a cover or plug of wood.

The smaller sizes of pipe can be lowered into the trench by means of sling-ropes handled by men on the banks, but the heavier

pipes require the use of a derrick and skids. The pipe to be lowered is placed on skids over the trench and then handled by blocks and falls attached to a derrick. The derrick may be a special pipe-derrick (Fig. 46) or one of the tripod type. The latter, when the upper block is provided with a catch by means of which the weight can be held in any position, will also be found very useful in setting hydrants, large valves, and raising large stones from the trench.

Small pipes should never be permitted to rest upon the bell alone; the bottom of the trench being carefully excavated in order that the pipe-barrel may have a continuous bearing. Mains of two feet and more in diameter are usually supported upon blocks and wedges, in which case the back-filling should be well compacted under and about the pipe. The blocks and wedges should be of sound wood, the blocks being from two to four feet long, eight to

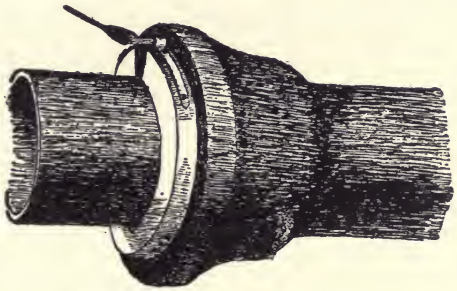


FIG. 47.—Gowing Pipe-jointer.

twelve inches wide, and three to six inches thick; the wedges, twelve to eighteen inches long, four to six inches wide, and three to four inches thick at one end, tapering to a thickness of one-half inch at the other, depending upon the size of the pipe and the nature of the foundation soil. Two sets, i.e. two blocks and four wedges, are placed under each pipe.

The pipe-joints should be carefully filled to the proper depth with yarn of good quality, care being taken to preserve the bore of the line continuous. The lead should be soft and of good quality and the joint completely filled at one pouring, the lead extending to the back of the groove cast on the interior of the bell. The old-time clay roll used in leading the joints has been superseded by the many patent pipe-jointers made of asbestos or rubber now on

the market, the use of which effects a saving in time and material. The yarning and calking should be accomplished by experienced men furnished with suitable tools for the purpose. The lead is usually melted in a furnace similar to the one shown in the accompanying illustration. The yarn, which should be free from tar, is driven tightly into the bell with a yarning-iron, and after the joint is poured the lead is compacted by a calking-iron. These irons are of the form shown by the illustrations, a set of tools consisting of a hammer, a cold-chisel, a yarning-iron, and

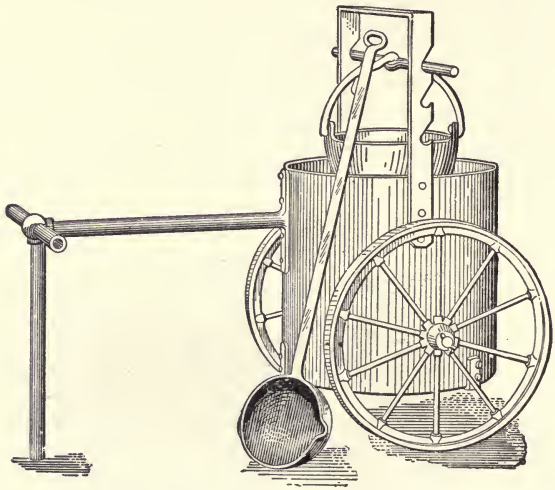


FIG. 48.—Lead Kettle and Furnace.

six calking-irons having calking edges varying in thickness from one-eighth to three-quarters of an inch. Superfluous lead is cut away with a cold-chisel.

Pipe may be cut when necessary by the use of a pipe-cutting chisel (see Fig. 49). The pipe to be cut should be supported by planks or joists and should be slowly rolled along the supports while the chisel is struck with a heavy sledge-hammer. A light cut should be first made entirely around the pipe. This cut is made deeper by successive operations, during whch the force of the blow upon the chisel is increased, until the pipe parts.

The manner in which the refilling of the trench is accomplished is dependent upon local conditions and the character of the material. When after-settlement is not desirable, resort must be made to

thorough ramming with suitable tampers, the earth being deposited in four- to six-inch layers, wet thoroughly, covered with a few inches of dry material and then well tamped. Care should be taken to consolidate the material at the edges of the trench, and the finished surface should be left slightly crowned at the center. Valve-boxes should be set with the covers at the original grade of the street-surface. Trenches in suburban streets or roads may be refilled more economically by depositing the material in water or by flooding the trench after the earth has been back-filled loosely in layers, but puddling or flooding should not be carried out with clay or earth containing a large proportion of that material. Previous

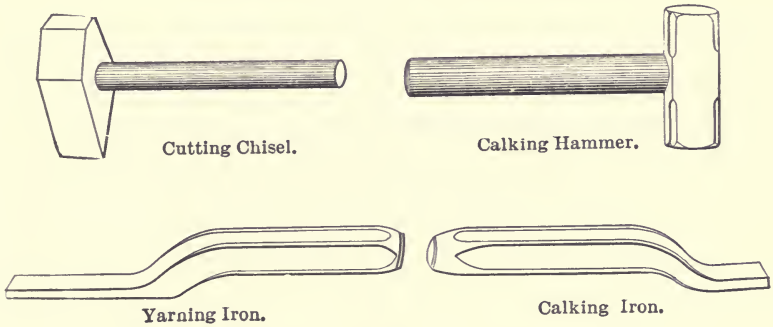


FIG. 49.—Pipe-laying Tools.

to the filling of a trench with water, sufficient earth should be placed about the pipe to prevent its displacement.

All materials should be inspected before acceptance on the work, and the spigot ends of cast-iron water-pipes should be re-examined before the pipes are lowered into the trench. Material rejected for any cause should not be allowed to remain upon the work, but should be removed immediately.

Materials.

The weight of cast-iron pipe to be used and the selection of valves and hydrants is governed more or less by the standards in use on the works, and a change in the type or make of valves or hydrants is not desirable unless experience has demonstrated that a departure from existing custom is advisable.

The use of cast iron for the mains of a distribution system is now the general practice, and the New England Water-works Association has developed a set of standard specifications for pipe and special castings of that material which serves as an admirable guide in the purchase of these supplies. With regard to the thickness of pipe for water-mains the committee of the above association reported: "The thicknesses given in Table No. 2 were computed by the following formula, which is one used in determining the thickness of pipe used on the Metropolitan Water-works, which supply water to Boston and other cities and towns within a radius of ten miles; Class A being for a static head of fifty feet, Class B for one hundred feet, etc., each class advancing by fifty feet. This formula provides factors for the deterioration of the pipe by time and other conditions, for the internal strain due to the static head and to water-hammer, but, as has been previously stated, other conditions must also be considered.

$$t=\frac{(p+p')r}{3300}+0.25,$$

in which t = thickness in inches;

p = static pressure in pounds per square inch;

p' = pressure in pounds allowed for water-hammer;

r = internal radius of pipe in inches;

3300 = $\frac{1}{5}$ tensile strength of cast iron, taken to be 16,500 pounds per square inch;

0.25 = allowance for deterioration by corrosion and other causes.

Values given to P′ as follows:

Diameter of Pipe.	P' in Pounds.
4-, 6-, 8-, and 10-inch	120
12- and 14-inch	110
16- and 18-inch	100
20-inch	90
24-inch	85
30-inch	80
36-inch	75
42- to 60-inch	70

"The committee does not recommend the classification of the weights upon the basis of static head, believing that to the engineer or superintendent should be left the final decision as to the thickness or weight of pipe suitable for the particular place in which it is to be used." *

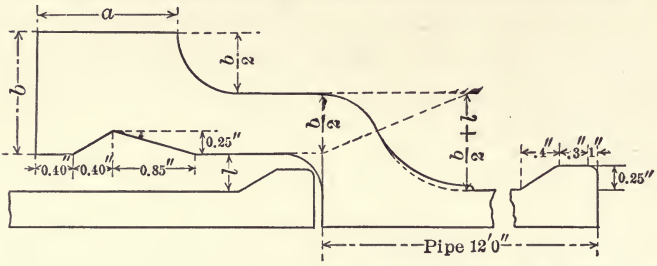
The following tables show the "General Dimensions of Pipes and Special Castings" and the "Standard Thicknesses and Weights of Cast-iron Pipes" adopted by the New England Water-works Association.

The hydrants in general use in small cities and towns are of the type known as "post" hydrants; the "flush" hydrant being rarely seen outside the large cities. Hydrants of the first class project above the surface, and the nozzles and upper portion of the valve-rod are above the level of the sidewalk; those of the second are entirely enclosed in masonry chambers built below the surface of the street. The details of the valve controlling the supply of water, its position when the hydrant is in use, and the operating mechanism vary widely in the several designs of post hydrants manufactured. The main valve consists of a number of leather disks held in place on the valve-rod by nuts and metal washers, or is made of metal, which in some designs is faced with leather or rubber. Hydrants supplied with water by a valve of the first type are usually termed "plug" or "compression" hydrants; those with metal or faced metal valves which move in a vertical plane, "gate" hydrants. The distinctive features of the two types are shown by the illustrations. A waste or drip is usually provided for the purpose of draining the water from the hydrant after use. The following details should be given consideration in the selection of the make of hydrant to be used:

Simplicity of operation;
Accessibility of valve and operating mechanism;
Durability;
Ease and cost of making repairs;
Drainage;
Area of water-way and loss of pressure due to friction;
Liability of water-hammer;
Liability of leakage when used frequently.

* Jour. N. E. Water-works Assoc'n, Vol. XVI, pp. 89–90.

General Dimensions of Pipes and Special Castings.



Nominal diameter, inches.	Classes.	Actual outside diameter, inches.	Diameter of sockets.		Depth of sockets.		a	b
			Pipe. inches.	Special castings, inches.	Pipe, inches.	Special castings, inches.		
4	A, C, E	4.80	5.60	5.70	3.00	4.00	1.50	1.30
4	G, I, K	5.00	5.80	5.70	3.00	4.00	1.50	1.30
6	A, C, E	6.90	7.77	7.80	3.00	4.00	1.50	1.40
6	G, I	7.10	7.90	7.80	3.00	4.00	1.50	1.40
8	A, C, E	9.05	9.85	10.00	3.50	4.00	1.50	1.50
8	G, I	9.30	10.10	10.00	3.50	4.00	1.50	1.50
10	A, B, C, D	11.10	11.90	12.10	3.50	4.50	1.50	1.50
10	E, F, G, H	11.40	12.20	12.10	3.50	4.50	1.50	1.50
12	A, B, C, D	13.20	14.00	14.20	3.50	4.50	1.50	1.60
12	E, F, G, H	13.50	14.30	14.20	3.50	4.50	1.50	1.60
14	A, B, C, D	15.30	16.10	16.35	3.50	4.50	1.50	1.70
14	E, F, G, H	15.65	16.45	16.35	3.50	4.50	1.50	1.70
16	A, B, C, D	17.40	18.40	18.60	4.00	5.00	1.75	1.80
16	E, F, G, H	17.80	18.80	18.60	4.00	5.00	1.75	1.80
18	A, B	19.25	20.25	20.40	4.00	5.00	1.75	1.90
18	C, D	19.50	20.50	20.40	4.00	5.00	1.75	1.90
18	E, F	19.70	20.70	20.70	4.00	5.00	1.75	1.90
20	A, B	21.30	22.30	22.50	4.00	5.00	1.75	2.00
20	C, D	21.60	22.60	22.50	4.00	5.00	1.75	2.00
20	E, F	21.90	22.90	23.00	4.00	5.00	1.75	2.00
24	A, B	25.40	26.40	26.60	4.00	5.00	2.00	2.10
24	C, D	25.80	26.80	26.60	4.00	5.00	2.00	2.10
24	E, F	26.10	27.10	27.10	4.00	5.00	2.00	2.10
30	A, B	31.60	32.60	32.60	4.50	5.00	2.00	2.30
30	C, D	32.00	33.00	33.00	4.50	5.00	2.00	2.30
30	E, F	32.40	33.40	33.40	4.50	5.00	2.00	2.30
36	A, B	37.80	38.80	38.80	4.50	5.00	2.00	2.50
36	C, D	38.30	39.30	39.30	4.50	5.00	2.00	2.50
36	E, F	38.70	39.70	39.70	4.50	5.00	2.00	2.50
42	A, B	44.00	45.00	45.00	5.00	5.00	2.00	2.80
42	C, D	44.50	45.50	45.50	5.00	5.00	2.00	2.80
42	E, F	45.10	46.10	46.10	5.00	5.00	2.00	2.80
48	A, B	50.20	51.20	51.20	5.00	5.00	2.00	3.00
48	C, D	50.80	51.80	51.80	5.00	5.00	2.00	3.00
48	E, F	51.40	52.40	52.40	5.00	5.00	2.00	3.00
54	A, B	56.40	57.40	57.40	5.50	5.50	2.25	3.20
54	C, D	57.10	58.10	58.10	5.50	5.50	2.25	3.20
54	E, F	57.80	58.80	58.80	5.50	5.50	2.25	3.80
60	A, B	62.60	63.60	63.60	5.50	5.50	2.25	3.40
60	C, D	63.40	64.40	64.40	5.50	5.50	2.25	3.40
60	E, F	64.20	65.20	65.20	5.50	5.50	2.25	4.00

Standard Thicknesses and Weights of Cast-iron Pipes 12 Feet in Length, Exclusive of Socket.

Nominal diameter of pipe.	Class A.		Class B.		Class C.		Class D.		Class E.		Class F.		Class G.		Class H.		Class I.		Class K.	
	Thickness of shell.	Weight per length.	Thickness of shell.	Weight per length.	Thickness of shell.	Weight per length.	Thickness of shell.	Weight per length.	Thickness of shell.	Weight per length.	Thickness of shell.	Weight per length.	Thickness of shell.	Weight per length.	Thickness of shell.	Weight per length.	Thickness of shell.	Weight per length.	Thickness of shell.	Weight per length.
	Ins.	Lbs.	Ins.	Lbs.	Ins.	Lbs.	Ins.	Lbs.	Ins.	Lbs.	Ins.	Lbs.	Ins.	Lbs.	Ins.	Lbs.	Ins.	Lbs.	Ins.	Lbs.
4	0.34	200			0.36	215			0.39	230			0.42	250			0.45	265	0.48	280
6	0.38	330			0.42	350			0.46	380			0.50	420			0.54	445		
8	0.42	475			0.48	530			0.53	575			0.58	640			0.63	690		
10	0.47	650	0.50	680	0.53	720	0.56	760	0.60	810	0.63	850	0.67	890	0.70	935				
12	0.49	810	0.53	855	0.57	910	0.61	970	0.65	1040	0.69	1100	0.73	1160	0.77	1220				
14	0.53	1010	0.57	1080	0.61	1150	0.66	1220	0.70	1310	0.75	1390	0.79	1460	0.83	1530				
16	0.55	1215	0.60	1300	0.65	1390	0.70	1490	0.75	1610	0.80	1710	0.85	1810	0.90	1900				
18	0.57	1400	0.63	1520	0.69	1660	0.75	1780	0.80	1910	0.86	2040								
20	0.60	1610	0.66	1760	0.72	1920	0.79	2090	0.85	2260	0.92	2420								
24	0.64	2050	0.72	2290	0.80	2550	0.88	2780	0.95	3000	1.03	3240								
30	0.71	2860	0.81	3230	0.91	3600	1.01	3950	1.10	4340	1.20	4700								
36	0.79	3800	0.90	4270	1.02	4840	1.13	5310	1.25	5900	1.37	6400								
42	0.87	4920	1.00	5560	1.13	6270	1.27	6970	1.40	7720	1.53	8360								
48	0.95	6130	1.10	6970	1.25	7920	1.40	8780	1.55	9740	1.70	10600								
54	1.03	7510	1.20	8600	1.37	9800	1.54	10900	1.72	12400	1.90	13500								
60	1.10	8900	1.30	10300	1.50	11900	1.70	13300	1.90	15100	2.10	16500								

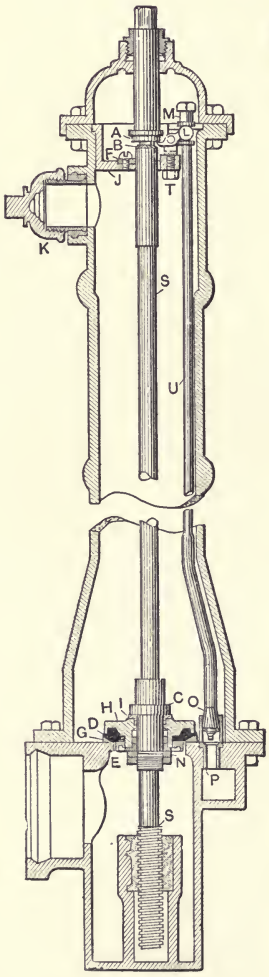


FIG. 50.
Section of Eddy Hydrant.

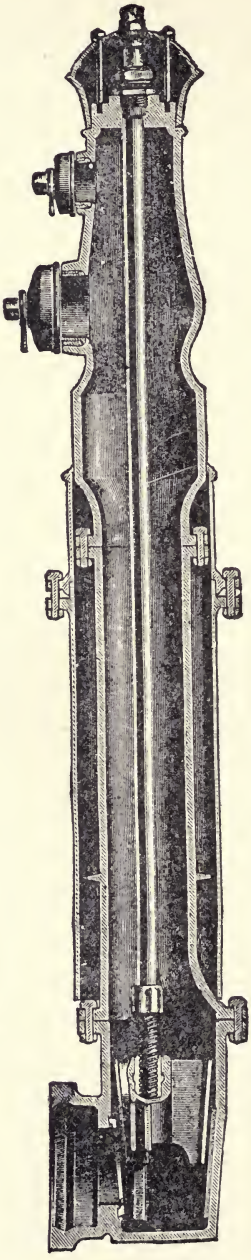


FIG. 51.
Chapman Hydrant.

FIG. 52.
Mathews Hydrant.

Post hydrants provided with two $2\frac{1}{2}$-inch hose nozzles and one steamer nozzle will usually prove satisfactory for general use. Frost-cases may usually be dispensed with, but are occasionally needed when hydrants are set in clay soils.

The valves used in street-mains are usually iron-body valves with bronze mountings, the wedge of which is operated by an inside screw. Valves made entirely of iron are not suitable for

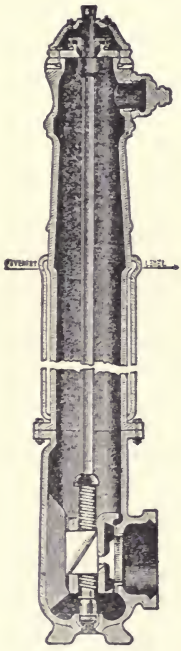


FIG. 53.
Ludlow Hydrant.

FIG. 54.
Corey Hydrant.

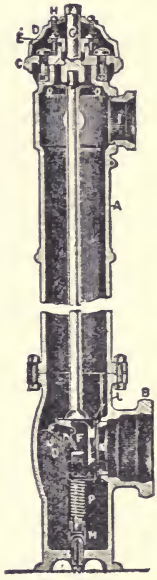


FIG. 55.
Walker Hydrant.

water-works use. The upper end of the screw or stem is capped with a square iron nut, and the screw is turned by an iron or steel rod with a socket on the lower end which fits over the nut, the latter being made accessible by the use of a cast-iron valve-box. The pattern of wedge used is the chief feature of difference in the various makes; the wedges of certain valves being cast as a single piece, and those of others consisting of two or more parts which are held together by the stem. Valves provided with a divided wedge are termed "adjustable-wedge" valves.

The smaller valves are usually placed in a vertical, and those larger than eighteen inches in size in a horizontal, position. In the latter cases gearing is required for the operation of the valve. The largest valves are provided with a by-pass by which the pres-

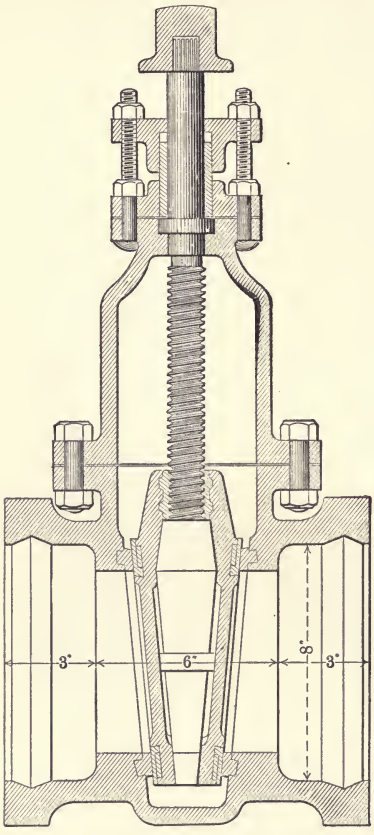


FIG. 56.—Section of Chapman Valve.

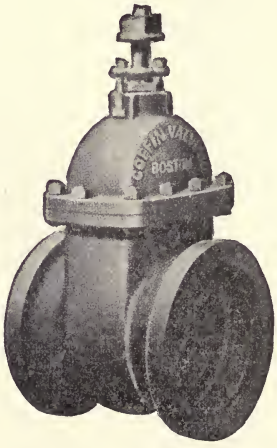


FIG. 57.—Coffin Valve.

sure on the sides of the wedge is equalized when the valve is opened, and are enclosed in masonry manholes.

Sectional extension valve-boxes are placed over the smaller valves. Three designs of boxes are shown. Boxes composed of sections which have no mechanical connection are preferable in streets where the traffic is heavy, as the boxes with sections having threaded joints are more easily broken by road-rollers and heavy teams.

Estimates of Cost.

Estimates of the cost of proposed work are based upon the prevailing prices of materials and the wages paid in the locality

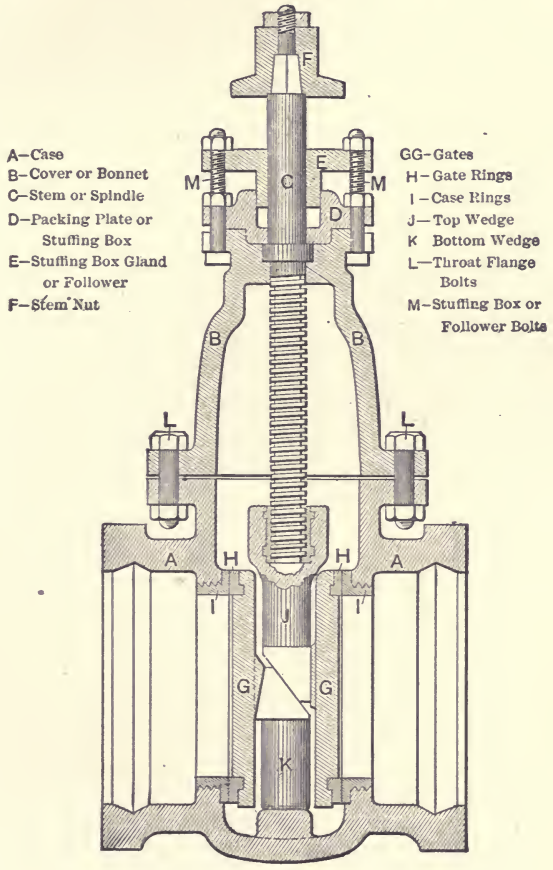


FIG. 58.—Section of Ludlow Valve.

where the work is to be done. The preparation of estimates of cost is facilitated by the use of tables or diagrams. The cost of cast-iron water-pipe is the principal factor in computations of cost of pipe-lines, and this cost is usually from $25.00 to $30.00 per ton, but often the quotations, particularly for the smaller sizes, exceed these figures. Fig. 61 shows the cost per foot at varying prices

per ton of 2000 lbs. of the several sizes of cast-iron water pipe from 4 to 24 inches in diameter, included in Classes A, C, and E of the New England Water-works Association. A similar diagram based upon the actual weights of pipe in use in

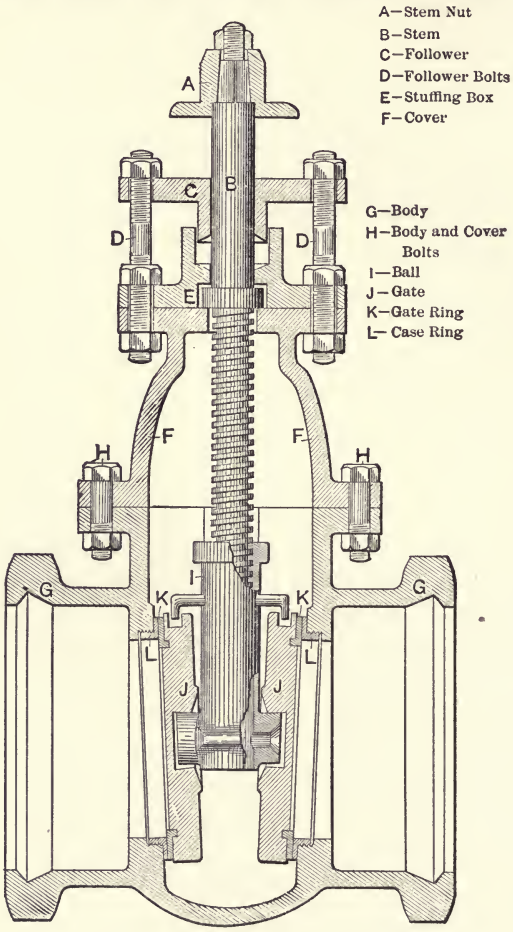


FIG. 59.—Section of Eddy Valve.

a given instance can be readily prepared and will often be found convenient in estimating. The value of the diagram will be increased if to the cost of the pipe per foot be added the cost per foot of pipe-laying, etc. The latter may be ascertained from figures obtained from actual work previously performed, and three lines may be drawn upon the diagram for each size of pipe: one

representing the total cost per foot when the excavation is considered easy, one when this is difficult, and a third when average conditions are encountered.

The cost of excavating, back-filling, and pipe-laying per linear foot of trench under the average conditions which obtain in small cities and towns in New England is about as follows:

Size of pipe, inches. . . .	4	6	8	10	12	14	16	18	20	24	30
Cost per foot, cents. . . .	18	21	25	30	35	42	50	60	70	95	$1.30

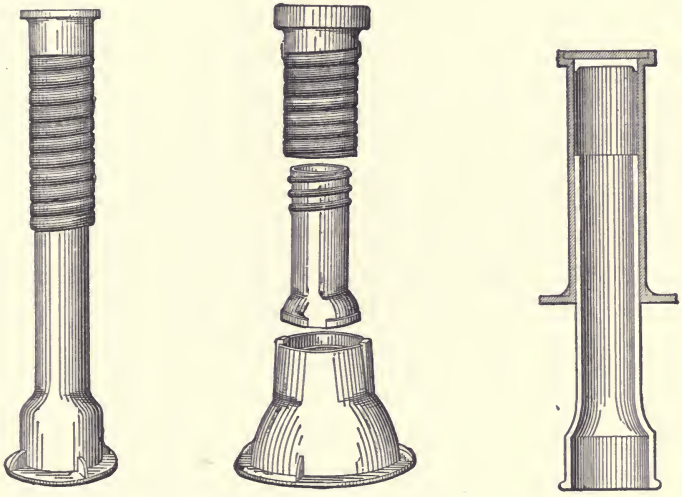


FIG. 60.—Valve-boxes.

The cost of rock excavation averages from $3.00 to $4.00 per cubic yard.

From 1¼ to 1¾ lbs. of lead are required for each inch of internal diameter for making joints in cast-iron pipe lines; i.e., the lead required for a 6-inch pipe-joint varies from about 7½ to 10½ lbs. The pig lead used for joints costs from 4½ to 5 cents per pound. The amount of yarn required for joints varies from 0.05 to 0.1 lb. per inch of internal diameter. The cost of yarn or jute is from 4 to 4½ cents per pound.

Valves for water-mains cost about as follows when purchased in small quantities:

Size, inches.	4	6	8	10	12	14	16	18	20	24
Cost.	$6 to $8	$10 to $13	$14 to $20	$22 to $28	$28 to $36	$44 to $54	$60 to $75	$80 to $100	$100 to $125	$140 to $180

Valve-boxes cost from $2.75 to $4.00 each.

Post hydrants with 6-inch connections, two 2½-inch hose nozzles and one steamer nozzle cost from $25.00 to $31.00 each. Frost-cases are from $2.00 to $3.00 additional.

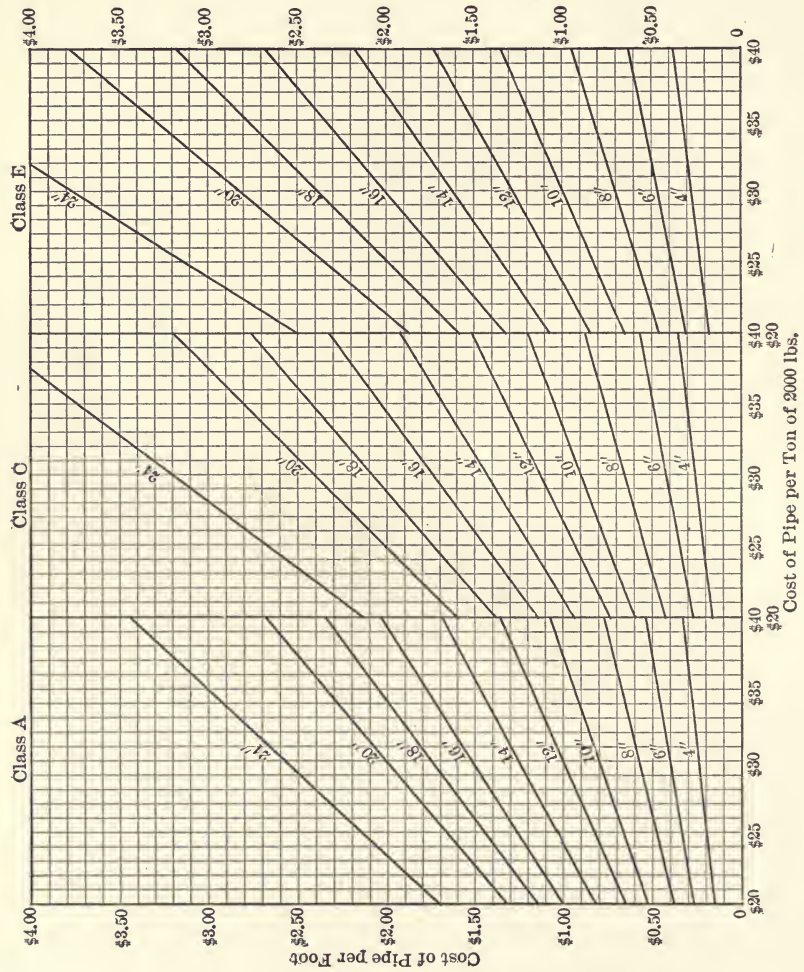


FIG. 61.—Cost of Cast-iron Pipe.

Special castings are either sold by the piece at varying rates of discount from established lists or by the pound at the market price for castings.

CHAPTER III.

SERVICE CONNECTIONS.

THE link between the street-main and the interior piping on the premises of the consumer is the service connection. As a measure of precaution against possible leakage due to poor workmanship,

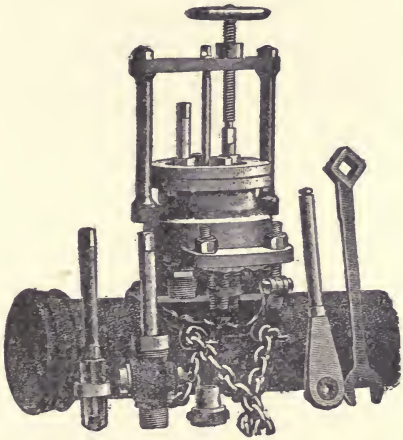


FIG. 62.—Hall Tapping-machine.

imperfect fittings, or the use of pipe of inferior grade, and unauthorized connections, made either with the mains or underground service-pipes, all service-pipes and fittings from the street-main to and including the stop and waste cock on the applicant's premises should be furnished, laid, and maintained by the water department. A proper charge may be made the consumer for the portion of the connection which is outside the limits of the street or way, and an additional charge, if such is the custom, for the tap and street connection.

The service connection consists of the tap or corporation cock in the main pipe, a lead gooseneck, a curb-cock with suitable box, a stop and waste cock on the consumer's premises, and all intervening piping.

The main pipe is tapped and the corporation cock screwed in place by a tapping-machine designed to operate with the main under pressure. There are several good machines for this purpose on the market, one of which is shown in Fig. 62.

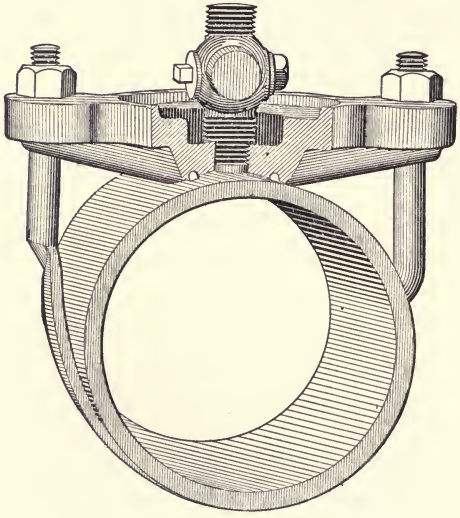
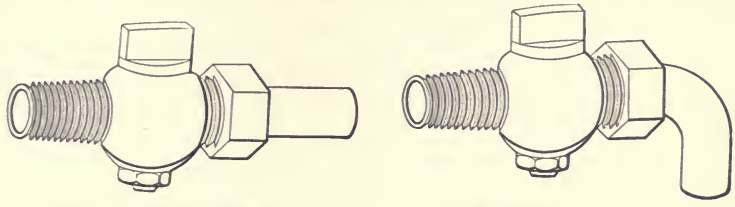


FIG. 63.—Service-clamp (Mueller Pattern).

The check on the tap and drill should be adjusted to allow the corporation cock to project through the pipe wall about one-half to three-quarters of an inch, otherwise the tuberculation, which takes place on the interior of the main more rapidly at this point as a result of the injury to the protective coating, will slowly but effectually close the opening. The corporation cock is usually of the same diameter as the service-pipe, but as it is inadvisable to make large taps in small mains, since a good contact between the threads on the cock and in the main is impossible and the resulting joint possesses little strength or rigidity, it is occasionally necessary to make several taps and combine the goosenecks when large service connections are laid. The use of several taps may be obvi-

ated by the use of a tapping-band or service-clamp, which insures that the corporation cock will be held firmly in place. These bands are drilled and tapped for the reception of the corporation cock, bolted about the main pipe, and the connection made in the usual



With Straight Coupling. With Bent Coupling.

FIG. 64.—Corporation Cocks.

manner. Leakage is prevented by the use of a rubber gasket, and cement or lead placed in a groove cast in the band.

As a general rule the smaller mains are tapped on top, and the

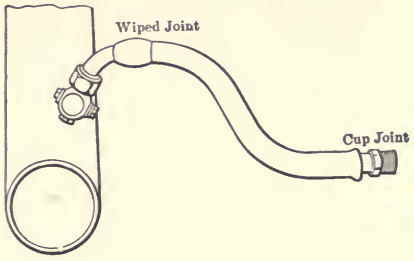


FIG. 65.—Gooseneck.

larger on the side. In the first instance corporation cocks with bent couplings are used, and in the second, those with straight couplings.

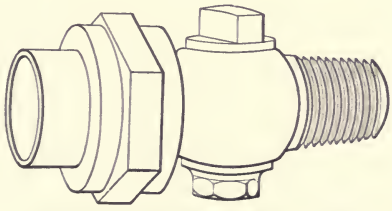
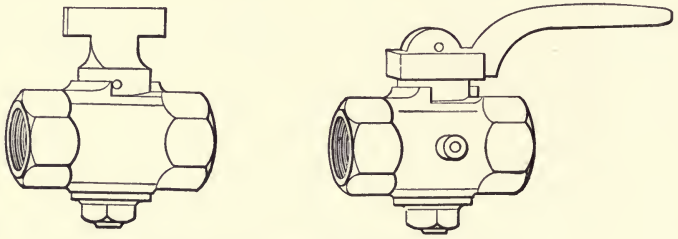


FIG. 66.—Corporation Cock with Lead Flange-coupling.

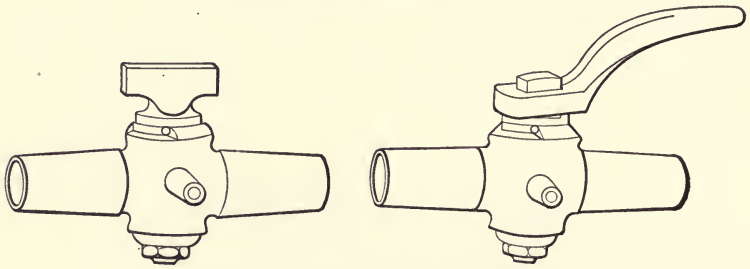
Unless lead pipe is used for the service, a lead gooseneck about two feet long is placed between the corporation cock and the service-pipe, care being taken that the upper portion of the

bend is not placed too high in localities where the ground freezes near the main pipe. The gooseneck is provided as a safeguard against excessive strain, due to the settlement of either main- or service-pipe resulting from excavations for sewers or drains.

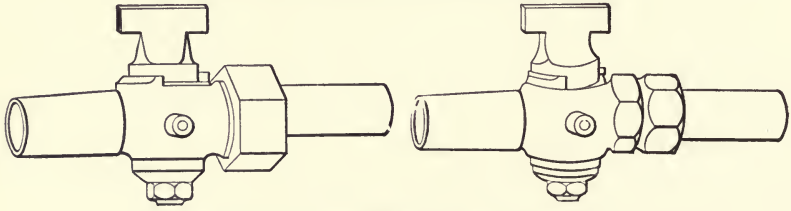
The joint between the corporation cock and the gooseneck may be made with solder, either a "wiped" or a "cup" joint being



Stop and Waste Cocks for Iron Pipe.



Stop and Waste Cocks for Lead Pipe.



Union-joint Stop and Waste Cocks for Lead Pipe.

FIG. 67.—Stop and Waste Cocks.

made, or the solder may be omitted and the connection made with a lead flange-coupling. The "wiped" joint presents a more workmanlike appearance, but the "cup" joint when properly made has given entire satisfaction and requires less labor and material. The lead flange-coupling consists of a special union in which the end of the lead pipe is spread and flattened with

suitable tools to form a gasket for the joint. The material costs about the same as that for the "wiped" joint, but the joint does not require the skilled labor necessary in the latter instance.

The corporation cocks and the stop and waste cocks should be substantially made, turned, and bored in a workmanlike manner, and should be of the type known as "round way." The materials entering into their composition and the proportions used at Providence, R. I., are as follows:*

80 lbs. copper;
6 lbs. tin;
3 lbs. zinc;
2 lbs. lead.

Mr. William R. Hill recommends a mixture of 88 parts refined copper, 6 parts tin, 3 parts zinc, and 3 parts lead with the pro-

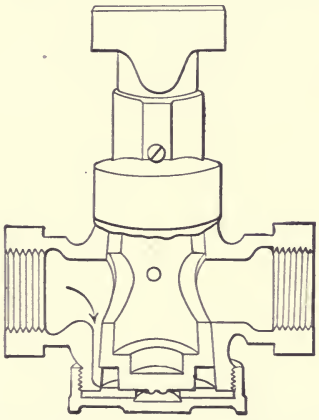


FIG. 68.—Section of Inverted-key Stop and Waste Cock.

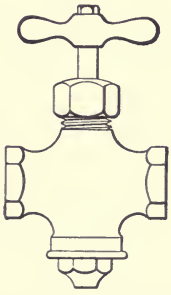


FIG. 69.—Compression Stop and Waste Cock.

viso that "if there is alkali or salt in the soil the zinc should be omitted on account of corrosion." Also that "the plugs should be lubricated by plumbago and Albany grease or vaseline. Beeswax and tallow should not be used, as it will get hard and make it difficult to open or close the valve."† One objection to the ordinary plug-cock is overcome in the "inverted-key" type.

* Jour. N. E. Water-works Assoc'n, Vol. XVIII, p. 24.
† *Ibid.*, Vol. XIII, pp. 37, 38.

The plug of this cock is inverted and the pressure upon the rod in opening or closing the valve tends to loosen the plug and facilitate its movement. The joint is kept tight by the pressure of the water from the street side, a by-pass admitting water under pressure to the under side of the plug. The stop and waste cock at the curb is usually provided with a tee handle, that in the basement, with a lever handle. A compression stop and waste cock, somewhat similar in appearance and operation to an ordinary faucet, has been recently introduced on the market, and may be used to advantage in some cases. For lead-pipe work the fittings provided with union joints are to be preferred, as their use affords a means of ready access to the interior of the pipe in case of stoppages caused by small fish, eels, or ice. The joints between the lead pipe and fittings may be made in any of the ways mentioned in the consideration of the joint between the corporation cock and the gooseneck.

The fixtures in any ordinary residence or building may be adequately supplied by a service connection of five-eighths or three-quarters of an inch in diameter, but hotels, business blocks, etc., often require services of larger size.

For ordinary service connections, pipes of wrought iron, which may be plain, tarred, galvanized, or lined with cement, lead, or tin, and of lead or tin-lined lead are in common use. Services larger than two inches in diameter may usually be more economically laid with cast-iron pipe. The three classes of pipe first mentioned are a source of considerable annoyance and expense in many systems, as some waters corrode the iron with great rapidity, the tar or galvanized coatings as at present applied merely postponing this action in a measure. As a result of the corrosive action the pipes are quickly filled with rust, fixtures and meters clogged, and the water rendered objectionable for laundry purposes.

Cement-lined iron pipe when properly lined often gives entire satisfaction, but has been found objectionable in some cases owing to the cracking and flaking off of the cement and the difficulty in obtaining rust-proof joints. This pipe has been successfully used in Brookline, Massachusetts, for more than twenty-five years, and the methods used in its lining have been described in

detail by Mr. F. F. Forbes in a paper read before the New England Water-works Association, from which the following extracts are taken:*

"Our departure from the usual custom begins with the buying; we specify that the lengths shall be about 16 feet long and also of standard weight. A pipe 18 or 20 feet long cannot be lined with the same degree of success which can be obtained with pipes somewhat shorter; and our experience has taught us that pipes about 16 feet long give the best results. After the pipes are delivered at our shop, we first straighten all which are much bent. The couplings are then removed, turned around and screwed on the other end, in order that there may be no trouble in putting the lengths together when lined. The pipes are then carefully examined in order to make sure that no defect exists in the welds or other parts of them. The next step is to run a cutting-tool slightly smaller than the inside diameter of the pipe through each length to remove all dirt, scales, and projections of iron from the welds. The pipes are now ready for lining.

"No sand should be mixed with the cement. Portland cement is not fit for this work, being too heavy and liable to fall from the sides of the pipes before setting. We have used the F. O. Norton brand with great success; but any good American natural cement which does not set too rapidly and is freshly ground can be used with confidence. It is very necessary to sift all the cement through a moderately fine sieve, as we find that even the best cements contain small pieces of unground rock and other substances which interfere seriously with the lining. It is also extremely important that the cement should be used quickly after wetting. When lining we have one man who does nothing but mix the cement, usually preparing enough for five or six pipes at one time and constantly working it over to keep it at the right thickness. If a little of the batch is left, we prefer to throw it away rather than to mix it with the next lot. The press used for filling the pipe is made by the Union Water Meter Company, of Worcester, Mass. in fact, they make the

* "Cement-lined Service-pipes." F. F. Forbes, Jour. N. E. Water-works Assoc'n, Vol. XV.

whole outfit, including cones, etc. It is necessary to fill the pipe entirely full of cement, and a little should be allowed to run out of the farther end. More of the cement will be pushed out by the cones, but this can be returned to the mixing-box and used again, with the exception of that from the last pipe filled from the batch, which is thrown away. In every case the cones are passed through the pipes twice. A handful of cement is pushed into the pipe before the cone enters the second time, and while it is being drawn through, the pipe is slowly revolved to keep the cone in the center of the pipe. We endeavor to keep the cones as near the center of the pipe as possible; we find, however, that the practical results are the same if the lining is quite uneven. The cones are thoroughly washed after each pipe is lined; before they are drawn through, a piece of pipe from 12 to 18 inches long is screwed to each end of the pipe to be lined, so that the lining at the end will be perfect. After the pipes have been lined from three to five days, or until the cement has sufficiently set, a thin gruel of cement is run through them. This is done by elevating one end of the pipe and pouring the gruel in from an ordinary watering-pot. A rubber cone is now drawn through, which leaves the inside of the pipe smooth and quite impervious to water. The ends are now reamed out to fit the composition ferrules and the threads cleaned. This completes the process, and the pipes are piled away for use. The number of feet of pipe of different sizes lined and grouted by one barrel of cement is as follows:

1 -inch pipe	700 feet
$1\frac{1}{4}$- " "	500 "
2 - " "	300 "

In 1898 the cost of labor, cement, etc., for lining 3000 feet of 2-inch pipe and 9000 feet of 1-inch pipe was as follows:

Labor preparing pipes	$65.79
" cementing	66.65
" grouting	22.66
" reaming	39.98
23 barrels cement @$1.10	25.30
Coal for heating shop	6.00
	$226.38

which gives the cost of lining 2-inch pipe 3.03 cents per foot, and cost of lining 1-inch pipe 1.5 cents per foot.

"Two men usually get the pipe ready for lining, but during the process of lining, six men are found to be the most economical number to use, distributed as follows: one man mixing the cement, one man filling the press and overseeing the work, one man working the press, one carrying the pipes to the press and from the press to the coning-frames, and two men, one at each end of the pipe, doing the coning. In one day these men will line from four to five thousand feet of pipe.

"A few words describing the ferrules we use at all joints. These are made of the best steam metal, five-eighths of an inch in diameter on the inside, the outside tapering slightly toward the end. They are of two kinds, called by us a double and a single ferrule. The double ferrules are used where the pipes are screwed together, and the single ferrules where the pipes are screwed into the sidewalk stops and connections at the main, which are made with a shoulder at the end of the thread to hold the ferrules in place.

"It has often been stated that the cement-lined pipe cannot be bent without injury. We find, however, that the pipes can be bent to any reasonable extent, without any damage to the lining, if this is done with care. We rarely use any elbows or tees. If we have to use a turn smaller than we think best to bend the pipe around, we use a short piece of lead pipe, perhaps a foot long, with a coupling on the end, and in making the connection use ferrules."

Tin-lined pipe is not extensively used owing to its cost, but when of good quality it possesses an advantage over other service-pipes, inasmuch as tin is rarely affected by potable waters.

The numerous reported cases of lead-poisoning resulting from the use of water drawn from lead service-pipes have prejudiced some consumers against them, and have caused water-works officials and others to doubt the advisability of the use of pipes of this material. Under certain conditions such use may endanger the health of a community, and it would be advisable to enlist the services of a chemist conversant with this matter before the adoption of lead as the material for the service-pipes of a system.

Investigations of the action of water upon lead pipe have been made in Massachusetts under the direction of the State Board of

Health, the results of which appear in the published reports of that board. The results of these investigations indicate that the corrosive action of the potable waters of Massachusetts upon lead is due principally to the presence in the water of comparatively large amounts of carbonic acid, and, in a measure, oxygen; the extent to which the dissolving action takes place being dependent chiefly upon the degree of hardness of the water. "The greater the hardness of the water, as compared with its free carbonic acid, the less effect did the carbonic acid have upon lead." * The following extracts from the published reports are of interest in this connection.

"Nearly all of the serious cases of lead-poisoning resulting from the use of water taken from public water-supplies (in Massachusetts) have occurred in those cities and towns which are supplied with ground-water; but surface-waters also act upon lead pipe, and though the quantity found in surface-waters drawn through such pipes has usually been small, there is, nevertheless, danger that injury to health may result even if the water is drawn from a surface source if lead pipe is used in its distribution." †

"A water which ordinarily acts but slightly on a lead pipe may, by some change in conditions, take up a much larger quantity of lead than under ordinary circumstances, and a change in the source of the supply of a city or town has been followed by a material increase in the action of the water upon lead service-pipes." ‡

"While the quantity of lead dissolved may be small, and a single dose might not seriously harm the user of the water, the continued use of water containing lead is harmful, because lead is a cumulative poison. The exact amount of lead which may be taken into the system without producing harm is not definitely known and may vary with different people, but it is known that the continuous use of water containing quantities of lead as small as .05 of a part per 100,000, or about $\frac{1}{33}$ of a grain per gallon, has caused serious injury to health." §

However, since many potable waters do not attack lead seriously, and in the case of some soft surface-waters, the presence in the

* Report of Mass. State Board of Health, 1900, p. 488.
† *Ibid.*, 1901, p. xxxi.
‡ *Ibid.*, 1901, p. xxxii.
§ *Ibid.*, 1898, p. xxxii.

water of certain mineral matters results in the formation of coatings on the interior of the pipe which serve to protect the metal, pipes of lead, or of iron lined with lead, are frequently used. As a measure of precaution in any case where lead or lead-lined pipes are in use, the water which has stood in the pipes overnight or for any considerable length of time should be wasted.

The quantity of water contained in service-pipes may be computed with the aid of the following table.

Internal diam. of pipe in ins.	½	⅝	¾	1	1¼	1½	2
Contents in gallons per 100 feet of pipe	1.02	1.59	2.30	4.08	6.38	9.18	16.32

The service-boxes in general use are of the telescopic pattern and usually consist of three parts: a base or lower section, an upper section, and a cover. The lower section is enlarged at the bottom in order that the curb-cock may be partially enclosed and also that resistance may be offered to the lifting action of frost. The upper section either slides over the lower or is connected to it by a threaded joint. One objection to the threaded joint is, that in some soils the entire box is lifted by frost action and cannot easily be driven back into place; another is that the upper section of a box of this kind is more liable to be damaged by teams. Circular covers are commonly used, but boxes set in brick walks should be provided with square or rectangular covers. The top of the box or cover should have a beveled edge, as this offers less obstruction to passers-by. The cover is held securely in place by a brass screw-bolt provided with a triangular or pentagonal head, as loose covers or covers held in place by bolts with square or hexagonal heads are easily removed by small boys and other unauthorized persons. This bolt either screws into the top section of the box or forms a part of a special clamp designed to hold the cover in place. In several designs for service-boxes another casting is added, the purpose of which is to hold the stop-cock in the center of the box with the tee head in a vertical position. Some boxes are provided with a short rod which is attached to the cock and held in the center of the box by lugs on the interior of the casing. Boxes of this type may be used occasionally to advantage in localities where the ground-water rises in a soft

material, with the result that the bottom of the box is often partly filled with mud. A disadvantage of the type is that it permits of too easy access to the shut-off by unauthorized persons.

As a general rule, the service connection should be laid at

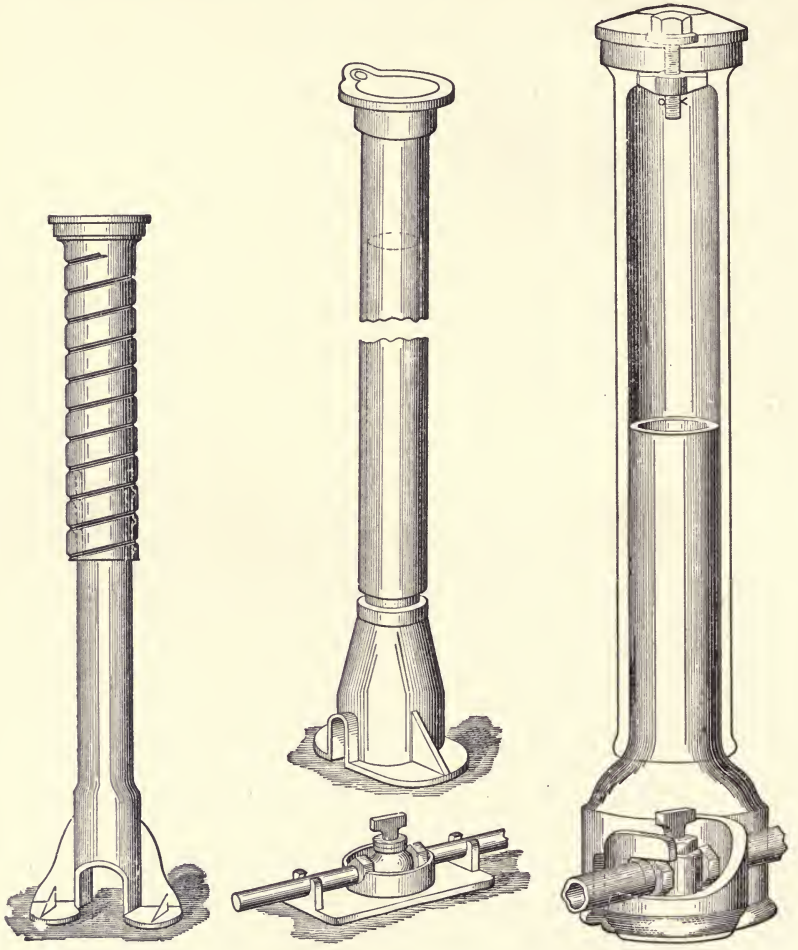


FIG. 70.—Service-boxes.

right angles with the street-line and below the depth ordinarily reached by frost. It should enter the basement of the building to be supplied about one foot below the level of the floor, as this location affords better opportunities for the protection of the pipe,

stop and waste cock, and meter, if meters are used, from freezing. Where pipes are brought through the foundation walls above the level of the basement floor, freezing is liable to occur on the street side of the stop and waste cock when the water is shut off at night in houses not provided with heat in the basement, and in others when the building is unoccupied temporarily, and the water not shut off at the curb. When lead pipe is used a small arched opening should be left in the wall as a precaution against injury to the pipe caused by settlement or sliding of the wall stones. The location of the consumer's stop and waste cock should be such that it is accessible at all times, and in no case should the shut-off be placed in coal-bins, ash-pits, or in places where it may be covered with wood or rubbish. The stop and waste cock at the curb should never be omitted, as it will be found useful in the enforcement of regulations regarding non-payment of rates, and for cutting off the supply from premises which are vacant. On paved streets the cock should be so placed that the service-box will be in the sidewalk next to the curbstone, and in suburban streets, so that the box will be located in the grass ground between the gutter and the walk, as a precaution against accidents resulting from persons tripping over the box or cover.

The water should be turned on and all joints tested under pressure before the trench is refilled, as the water escaping from an imperfect joint will, especially in sandy soil, with the aid of particles of earth, rapidly cut away the pipe or fitting. Many persons object to the use of one trench for both water and sewer connections on the ground that leakage from the service connection may enter the sewer undetected. However, a leak of sufficient size to be detected by the appearance of water on the surface of the street where escape into a sewer is impossible will, if such escape is possible, usually make its presence known to a water-taker by a disagreeable roaring or hissing sound from the interior piping. Nevertheless, in cases where service-pipes have previously been laid, it is advisable to make a separate trench for the sewer connection as a precaution against injury to the service due to settlement, particularly when the sewers are laid by contractors or individuals.

The prices of materials required for service connections depend upon the quantity purchased, its quality, and the discounts from list prices in force at the time purchases are made.

Tapping-machines including drills and taps for ordinary service work cost from $60.00 to $125.00.

Corporation and stop and waste cocks when purchased in small quantities cost about as follows:

Style.	Size.		
	¾"	1"	1"
	$	$	$
Corporation cocks with couplings	0.55 to 0.85	0.80 to 1.05	1.20 to 1.60
Ordinary round-way stop and wastes for iron pipe		0.55 to 0.85	0.85 to 1.25
Inverted key stop and wastes for iron pipe		1.35	2.25
Compression stop and wastes for iron pipe		1.40	1.75
Ordinary round-way stop and wastes for lead pipe	0.60 to 0.75	0.80 to 0.90	1.00 to 1.50
Ditto with union joint	0.75 to 0.90	1.00 to 1.10	1.25 to 1.75
Inverted key stop and wastes for lead pipe	1.15	1.35	2.25
Lead flange stop and wastes for lead pipe	1.55	1.80	2.90
Inverted key with lead flange connections	1.95	2.20	3.50
Compression stop and wastes for lead pipe	1.00 to 1.15		

Service-boxes cost from 65 cents to $1.80.

Cost of Galvanized-iron Service-pipe.

Size, inches	¾	1	1¼	1½	2
List price per foot	$0.11½	$0.16½	$0.22½	$0.27	$0.36
Net price per foot	0.04 to 0.05	0.06 to 0.07½	0.07½ to 0.09	0.09 to 0.11	0.12 to 0.15

Cost of Lead- and Tin-lined Iron Service-pipe.

Size, inches	¾	1
Lead-lined, per foot	$0.14	$0.21
Tin-lined, per foot	0.24	0.32

The cost of cement-lined iron pipe is about as follows when made by the department. This pipe cannot be purchased at present from dealers in service supplies.

Size.	Approximate cost per foot.
1 inch lined to ¾ inch.	$0.09
1¼ " " " 1 "	0 12
1½ " " " 1¼ "	0.15
2 " " " 1¾ "	0.18

Lead pipe costs from 5 to 5½ cents per pound.

The weight per foot of lead pipe is governed in a measure by the pressure to which the pipe is subjected, and also by the

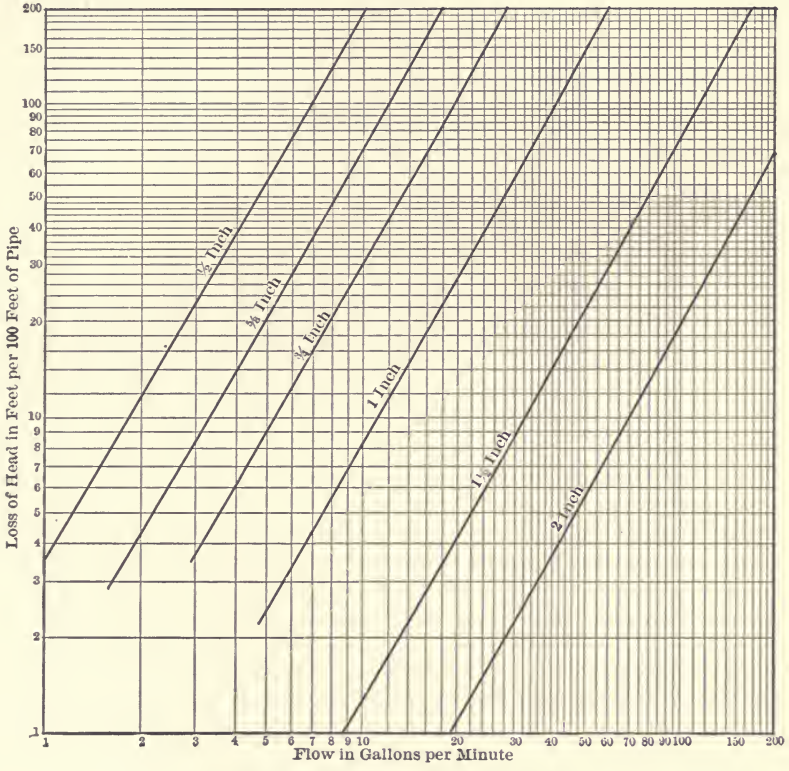


FIG. 71.—Discharge of Service-pipes.

standard weights supplied by the manufacturers. The weights given in the following table are sufficient for ordinary water-works purposes. The light pipe may be used on gravity works under a low head, the heavy for general purposes.

WEIGHTS OF LEAD PIPE PER FOOT.

Internal diameter, inches	½	⅝	¾	1	1½
Light, pounds	2	2½	3	4	6
Heavy, pounds	3	3½	4½	6	9

Solder costs from 20 to 25 cents per pound. The following quantities are usually allowed for "wiped" joints.

Diameter of pipe, inches	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	1	$1\frac{1}{2}$
Pounds of solder	$\frac{3}{4}$	1	1	$1\frac{1}{4}$	$1\frac{3}{4}$

Very little solder is required for "cup" joints.

Tin-lined lead pipe costs from $9\frac{1}{2}$ to 10 cents per pound.

The cost of the labor required for excavating, back-filling, and pipe-laying, including the tapping of the main, averages from 10 to 25 cents per foot.

It is seldom necessary to make any computations of the discharge of service-pipes, since an ample supply is usually furnished through any of the sizes of pipe ordinarily connected to a system. Such computations are sometimes made when buildings are located at a distance from the street-mains and at an elevation where the pressure is low. Fig. 71 has been prepared from the figures given in Weston's Tables, and may be used for long lines of pipe having a very smooth interior surface similar to lead.

CHAPTER IV.

METERS.

THE meters in general use for measuring the water furnished consumers are of the positive type. Meters of this type measure and automatically record, with varying degrees of accuracy, the amount of water which has passed through them, and are divided into several classes, known as reciprocating, rotary, oscillating, and disc piston meters, depending upon the type and mode of operation of the piston which is displaced by the passage of the water. Meters of the first class are provided with pistons or plungers which operate in a manner somewhat similar to those of a reciprocating pump. Those of the second class are provided with either two pistons of irregular form which revolve about fixed centers within an oval chamber, much like the runners in a rotary pump, or a single-toothed piston, which moves in a cylindrical chamber, the interior surface of which is provided with a number of projections and indentations. In the latter case the piston does not rotate about a fixed center as in the previous instance, but moves in such manner that the projections on and recesses in the piston and walls of the casing form a series of measuring-chambers. In meters of the fourth class mentioned the piston has a disc form and moves with an oscillating or wabbling motion, in its movement dividing the chamber alternately into receiving- and discharging-spaces. A radial slot cut in the disc embraces a fixed plate or wedge set vertically within the chamber and which prevents rotation of the disc. The oscillating piston meters operate in a somewhat similar manner.

For ordinary purposes the pistons or discs are made of hard rubber, which in some makes is reinforced with metal. Each complete revolution of the piston or disc permits the passage

through the meter of a quantity of water equal to the contents of a chamber or series of chambers of known capacity. A spindle attached to the piston or disc engages a train of gearing by which the movement of the piston or disc is transmitted to a dial or series of dials. The quantity of water which has passed through the meter is thus registered upon the dials in either cubic feet or gallons.

The smaller meters are usually provided with strainers located

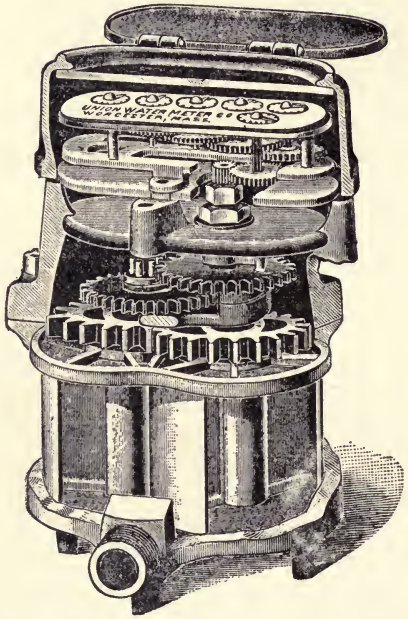


FIG. 72.—$\frac{5}{8}$-inch Union Rotary Piston Meter.

within the base and forming a part of the meter. Similar strainers are placed in some of the larger meters, but in general an additional fitting known as a fish-trap is set in connection with the meters of large size. Where the water carries considerable grit in suspension, or if small fish are often present in the pipes, the use of an independent fish-trap or grit-chamber is often advisable with the smaller meters, since the former appliances are more readily cleaned.

The type of meter to be used in any particular case depends to a considerable extent upon local conditions: the size of ser-

vice, amount of water consumed, pressure, the presence of sand, or other sediment in the water which would cause excessive wear of the casing and moving parts, the accuracy of registration desired, etc. In the selection of a meter the following points should be given consideration:

1. Accuracy.
2. Sensibility.
3. Durability.
4. Obstruction to flow.
5. Accessibility of working parts.
6. Liability of stoppage.
7. Cost of repairs.
8. Extent of damage as a result of freezing.
9. Capacity.
10. Cost.

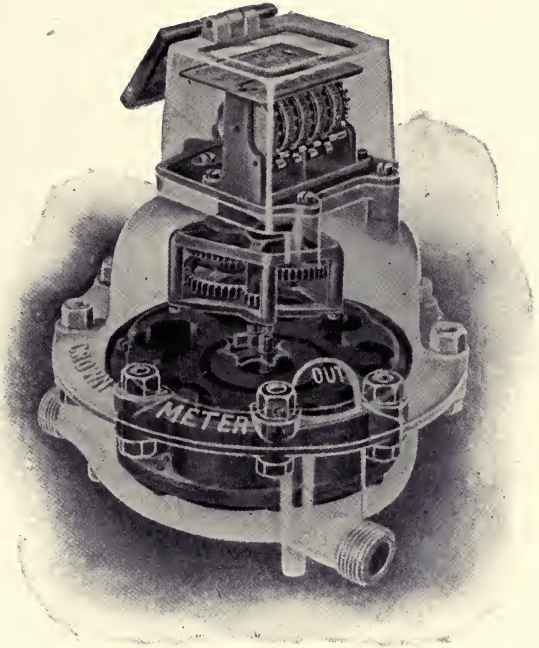
Service-meters of the usual size, five-eighths to three-quarters of an inch, when new should not over-register more than one per cent, and on varying rates of flow should register approximately the following percentages of the quantities of water actually passed through them:

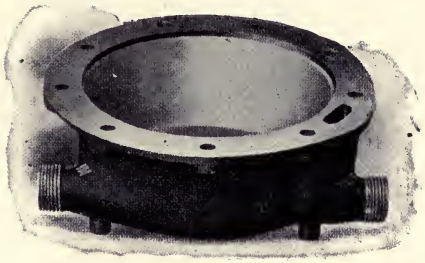
Flow in cubic feet per hour.	Per cent registered.
0.5	50
1.0	90
6.0	97
15.0 to maximum capacity	98–99

The maximum capacity of service-meters is given by the different manufacturers as follows:

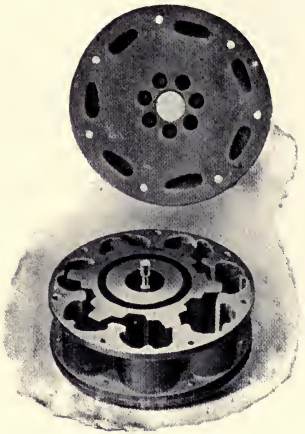
	Cubic feet.		Gallons.	
Size inches.	Per minute.	Per hour.	Per minute.	Per hour.
$\frac{5}{8}$	2	120	15	900
$\frac{3}{4}$	4	240	30	1800
1	8	480	60	3600

Where the detection and prevention of waste are of importance, meters should be tested for their sensibility to small rates of flow. This factor may be determined by testing a meter under a flow just sufficient to cause registration and noting the quantity passed and the percentage registered in a given time. In many cases this sensibility decreases to a large extent with use and tests





Base.



Cylinder, with Piston.

FIG. 73.—Crown Meter.

of this nature are of more value when made in connection with the durability test. The test for durability is made by passing through the meter a quantity of water equivalent to the consumption of an average family during five or ten years, approximately 100,000 cubic feet, after which the meter is tested for accuracy and sensibility. This test is usually made only in the comparison of meters of various types or makes as a guide to the selection of the type or make of meter best adapted for use on the works.

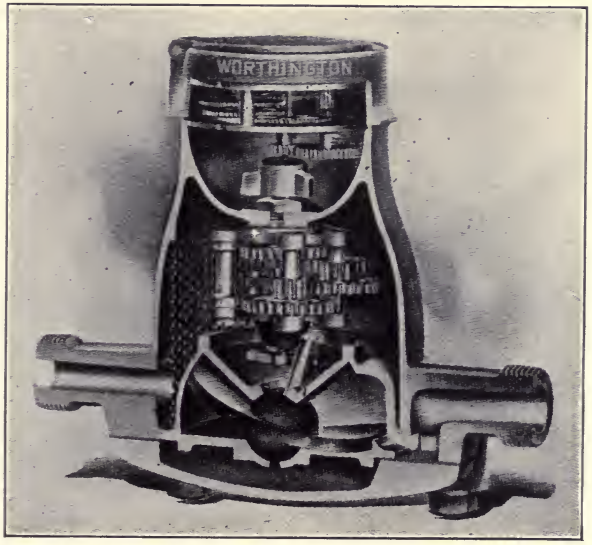


FIG. 74.—Section of Worthington Disc Meter.

The obstruction to the flow of water through a service connection due to the introduction of a meter is at times of importance, since it may be necessary in a given instance to use a meter of larger size than would be the case if a type or make of meter causing a less loss of head was selected. This consideration is of particular importance where meters are placed in pipes which supply water to motors or for purposes of fire-protection. The published results of tests made by Mr. J. W. Hill, C. E.,* of twelve five-eighths-inch meters and one three-quarter-inch meter of different makes, and by Mr. J. Waldo Smith, C. E.,† of six five-eighths-

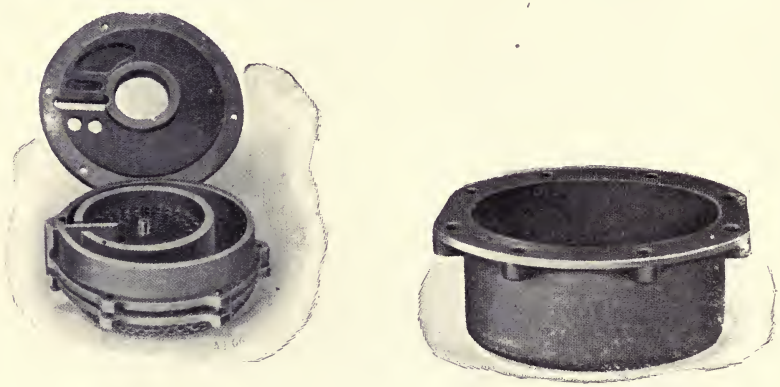
* Trans. Am. Soc. C. E., Vol. XLI, p. 407.
† *Ibid.*, p. 377.

inch meters, indicate that the loss of head caused by meters of the size tested is about as follows:

Flow in cubic feet per minute.	Loss of head in pounds per square inch. Tests by J. W. Hill.	Tests by J. W. Smith.
2.0	13 to 30	8 to 33
1.5	7 to 17	5 to 18
1.0	4 to 13	2 to 8
0.5	2 to 4	1 to 3

The meters included in the above table were all of the positive displacement type.

As a general rule, the size of meter to be used in a given instance should be based upon the amount of water used, and the rate of



Cylinder, with piston. Base.

FIG. 75.—Cylinder, Piston, and Base of AA Empire Meter.

flow, rather than upon the size of the service connection in which the meter is to be placed. A ⅝-inch meter will usually prove satisfactory for domestic service. For this purpose meters of the disc type are commonly used in preference to those of the piston pattern. Aside from the durability, permanence of registration, and economy in first cost and maintenance of disc meters, meters of this type possess a peculiar advantage over other types as a leak-detector. Mr. Hill * found that a noticeable vibration of the discs and attached spindle in the meters of this pattern tested occurred with rates of flow due to small leaks in the ser-

* Trans. Am. Soc. C. E., Vol. XLI, p. 350.

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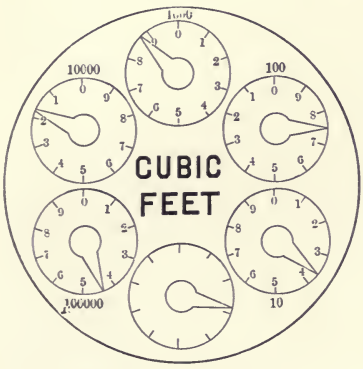
vice-pipe and fittings which were not sufficient to cause registration.

The following prices are given in the manufacturers' lists:

Size of meter.	Disc pattern. Cost.	Piston pattern. Cost.
$\frac{5}{8}$ inch.	$ 8.00	$10.00 to $12.00
$\frac{3}{4}$ "	12.00	15.00 " 21.00
1 "	16.00	20.00 " 30.00
1$\frac{1}{2}$ inches	30.00	40.00 " 50.00
2 "	50.00	60.00 " 65.00

A considerable reduction from the above prices may usually be obtained when large numbers of meters are purchased.



Round reading register.

Straight reading register.

FIG. 76.—Meter-dials.

Meters should be tested for accuracy before being placed in service, and, where meters are used to a large extent, suitable testing apparatus should be provided for this purpose. An independent supply-pipe should be laid from the street-main to the testing-room, a bench, connecting pipes, and fittings, pressure-gages, discharge-orifices from $\frac{1}{64}$ to $\frac{1}{2}$ inch diameter, and a measuring-tank provided. This tank may be calibrated and provided with a glass-tube gage with a scale indicating the contents of the tank corresponding to the height of the water in the tube; or the tank may be placed upon a platform scale, the water weighed, and the contents of the tank in cubic feet or gallons computed or read from tables prepared for that purpose. This latter method of testing is that generally adopted.

The meter-testing apparatus of the Concord, N. H., water-works is shown by the accompanying illustration. The Bureau of Standards at Washington has recently installed a meter-testing apparatus consisting of a bench, a Gow meter-clamping device, meter support, multiple-delivery cock with twelve orifices from $\frac{1}{64}$-inch to 2 inches in diameter, a tank of 64 cubic feet capacity, and a 5000-pound Fairbanks scale. The cost of the equipment was about $350.00.

The Gow clamping device and the various patterns of multiple-delivery cocks are almost indispensable in cases where large num-

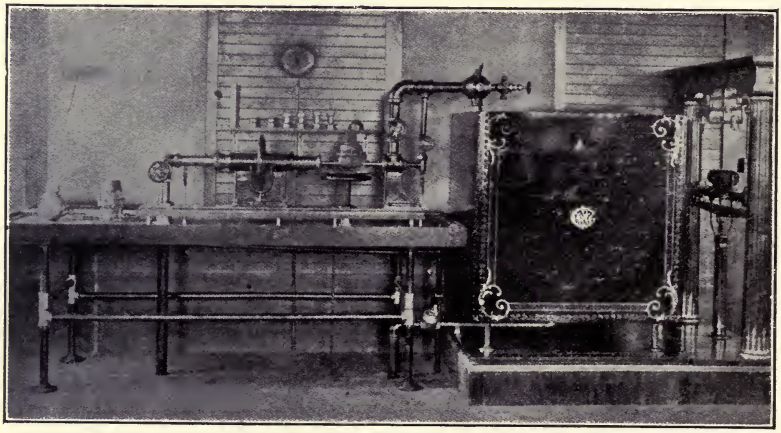


FIG. 77.—Meter-testing Apparatus, Concord, N. H., Water-works.

bers of meters are tested, owing to the saving in time effected by their use. Where but few meters are tested, short lengths of rubber hose fitted with suitable couplings may be used to connect the meter with the piping. The rate of flow may be varied by orifices drilled in brass screw-plugs, or in brass caps which are placed on the end of the outlet-pipe, or thin brass plates may be drilled and set in unions or couplings. A tight cask may also be used in place of a special tank.

The orifices for the regulation of the discharge should be placed on the outlet side of the meter, and the water should be discharged into the measuring-tank at an elevation somewhat higher than the meter. The use of an independent supply-pipe for the meter-testing apparatus insures a more uniform pressure during tests.

Before starting a meter test, water should be allowed to pass through the meter for a short time or until the air in the meter is expelled and the moving parts work freely. Care should be taken to turn the water on slowly, otherwise the disc or registering mechanism may be injured.

The meter should be stopped when the indicator of the one-cubic-foot dial is at the zero-point, the scale balanced, and the weight recorded. The flow is then started, time noted, and 5 to 10 cubic feet passed at the maximum rate. During the test the readings of the pressure-gages on the inlet and outlet sides of the meter are observed and the loss of head recorded. The flow should be stopped when the indicator of the one-cubic-foot dial is at the zero-point, time noted, quantity passed noted, and weight observed. The method of procedure is similar at the smaller rates of flow except that a smaller quantity of water is passed through the meter. During tests with the smallest orifices the meter should be examined for leakage.

For convenience, tables should be prepared which show the quantity of water in cubic feet corresponding to the weight of water found in the tank. The weight of a cubic foot of water varies with its temperature, but for ordinary work the weight is taken as 62.5 lbs. In some works the tables are based on the weight at 70° F. or 62.3 lbs.

WEIGHT OF ONE CUBIC FOOT OF WATER.

Temperature, degrees Fahrenheit.	Weight, pounds.
40	62.42
45	62.42
50	62.41
55	62.39
60	62.37
65	62.34
70	62.30
75	62.26

The accuracy of the meter, or the percentage of the actual amount of water passed which is registered on the dials, is obtained by the following formula:

$$\text{Percentage registered} = 100\left(\frac{\text{Quantity indicated by meter}}{\text{Quantity in tank}}\right).$$

Data with regard to the meters tested, the details of the tests made, and the results obtained should be preserved in books or on cards for reference.

In setting a meter, care should be taken that the meter is secured firmly in place and that it is level. Red or white lead should not be used in making joints, and the service-pipe should be flushed before the meter is connected. If there is a liability

Size and Style 3/4" Empire a-a-x Made by National Meter Co. Mfg. No. 271.681

Style of Register Straight Reading Limit One million Purchased Nov. 16 - 1900

Cost $15.75 Tested Nov. 21st 1900 By Phillips

RESULT OF TEST WHEN NEW.

Inlet	Duration of Test	Readings		Flow		Flow per Min.		Registered to Actual Flow	Pressure		
		Start	Stop	Reg.	Act.	Reg.	Act.		Inlet	Outlet	Loss
Ins.	Min.	Cu. Ft.	Cu. Ft.	Cu. Ft.	Cu. Ft.	Cu. Ft.	Cu. Ft.	%	Lbs.	Lbs.	%
1/8	9	000	2	2	2	0.22	0.22	100	50	50	00
1/4	5	2	7	5	5	1.	1.	100	50	45	10
3/4	4	7	22	15	15	3.75	375	100	50	5	90

Installed Nov 22 1900 At 314 N. Holliday St Ser. No. 20096

For W. W. Towson & Co.

Remarks

FACE.

REPAIRS

Date	Cost		Test Recd.	When Sent Out	Nature of Repairs	Cu. Ft. to date
1901 12-17	3	62	Hung up	100	New piston & facing off. (Frozen)	10000

REVERSE SIDE.

FIG. 78.—Card for Meter Tests, Baltimore Water Department. (*Eng. News*, vol. XLVIII, p. 357.)

that hot water may reach the meter from the interior piping, a check-valve should be provided near the outlet.

Meters are placed in basements, manholes, or boxes, the location and method of protection being dependent upon local conditions. The meter should always be placed on the house side of the curb stop and waste cock, and valves should be placed on each side of large meters in order that they may be accessible for inspection or repair.

Meters placed in basements are enclosed in boxes of wood or brick, which are filled with sawdust, mineral wool, or other suitable material if any possibility of freezing exists. Meters

placed outside buildings are set in manholes or boxes. Large meters should be enclosed in masonry vaults or manholes in order that the meter, fish-trap, and controlling-valves may be accessible without digging. Service-meters are placed in large boxes of wood provided with iron covers, one or more lengths of sewer-pipe, or a light cast-iron box, the meter being set near the surface in order that the dials may be easily read, with vertical inlet- and outlet-pipes enclosed within the box; or the meters are provided with dial extensions and are set at the same grade as the service connections, the meter and dial extension being

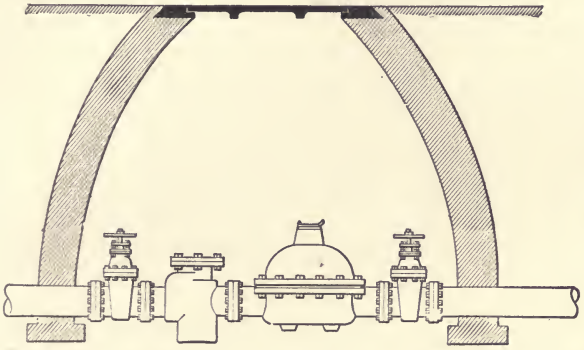


FIG. 79.—Manhole Setting for Large Meters.

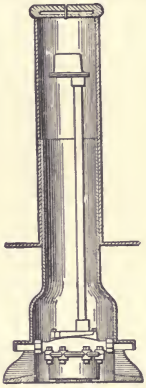


FIG. 80.—Volkhardt Meter-box.

enclosed in a small iron box somewhat similar to a street or service-valve box. The use of dial extensions is objectionable, as the friction of the registering mechanism is increased and the sensibility of the meter often seriously impaired by the addition of a dial extension.

Large numbers of service-meters have been set during recent years by the city of Cleveland in sewer-pipe, the method of setting at present in use being shown in Fig. 81. The meter is protected from frost by a tight cover of wood treated with preservative chemicals, which is placed below the outside iron cover to prevent the circulation of air about the meter. This wood cover consists of a circular rim, which is calked in place with oakum, and a small cover which is opened when the meter is read. An extra foot of sewer-pipe is used where the danger of freezing requires additional precaution. The experience with meters

set in this manner during the unusually severe winters of 1903 and 1904 has been very satisfactory.

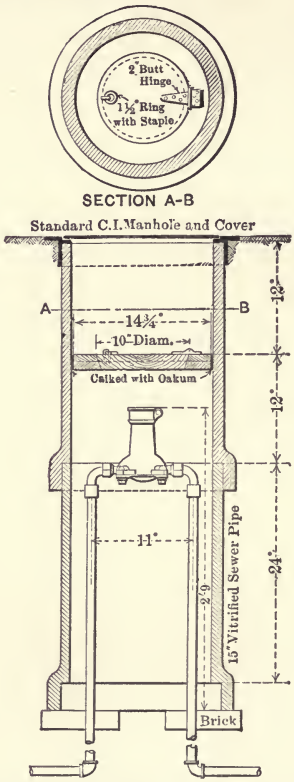


FIG. 81.—Setting for Water-meters, Cleveland, Ohio.

The cost of setting service-meters in basements is found by different water departments to be about as follows:

Taunton,	Mass.	$2.50
Watertown,	"	1.00
Brookline,	"	2.00
Springfield,	"	1.50 to 2.00
Fitchburg,	"	1.25
Lawrence,	"	1.25
Wellesley,	"	1.00

The cost of setting $\frac{5}{8}$-inch service-meters in Cleveland, Ohio, during 1903 is given in the following table:*

* "The Meter System in Cleveland." Edward W. Bemis, Proceedings of Am. Water-works Assoc'n, 1904.

Basement settings.		Brick vaults.		Sewer-pipe settings.	
Brick	$0.12	350 brick	$2.45	Sewer-pipe	$1.46
Cement	0.05	Cement	0.38	Frost-cover	0.18
Cover	0.30	Iron ring and cover	3.21	Iron ring and cover	1.42
Fittings	0.25	Fittings	0.50	Pipe and fittings	0.55
Material	$0.72	Material	$6.54	Material	$3.61
Labor	3.23	Labor	2.92	Labor	4.01
Total	$3.95	Total	$9.46	Total	$7.62

"During the year 1857 ⅝-inch meters were set in brick vaults, 3,174 in basements, and 9,378 in sewer-pipe." The item of labor for setting in brick vaults is comparatively low per meter, since more than one meter was often set in a single vault.

The estimated cost of setting ⅝-inch meters in the improved setting in 1904 is as follows:

Sewer-pipe	$1.84
Frost-cover	0.47
Cross-brace	0.04
Ring and cover	1.42
Pipe and fittings	0.55
Material	$4.32
Labor	4.51
Total	$8.83

Meters on domestic services are read quarterly, and those on commercial services monthly, in many works; in others, all meters are read once a month. This latter method is, as a rule, to be preferred, as the liability of loss due to the stoppage of a meter or the breaking of any of the parts is thereby diminished. When such occurrences take place, the bill may be adjusted on the basis of the meter registration for the previous period, the registration for the period after the meter has been repaired or replaced, or an average of the two.

Whenever a decrease in the amounts registered between meter readings is noted, unless the variation can be satisfactorily accounted for, the meter should be removed for testing. Service-

meters which have passed from three to five hundred thousand cubic feet of water usually under-register and should be removed for testing and repair. Such meters are often repaired by changing the gears in the registering apparatus. If the piston and casing are much worn, or are grooved or cut, the defective parts may be made smooth and the casing turned down in a lathe.

Meters will often pass a large proportion of their rated capacity when entirely stopped. Accumulations of rust in the upper portion of the casing and about the gear-train interfere with the proper operation of the latter. As the movement of the displacing member and the gears are usually audible, the meter-readers should note whether the meter is working or not at the times readings are taken. All meters, sediment-chambers, and fish-traps should be cleaned at intervals, the length of time between cleanings being determined from local experience.

The reading of the meter-dials is at times rendered difficult by the presence of moisture on the under side of the dial-glass. This condensation may be often prevented by placing a piece of cloth or heavy paper between the lid and the glass cover, or by covering the dial-box with burlap.

As a rule the boxes containing outside meters should not be opened in extremely cold weather.

The accuracy of service-meters in use may be approximately determined without removing the meters for testing by a method in use in Newton, Mass.* The occupants of a house supplied by a metered service having been notified to use no water, the meter is read by an inspector and 1 cubic foot of water passed through an orifice one-quarter of an inch in diameter drilled in a brass cap which is attached to a convenient faucet. The water is caught in a pail of known capacity. After the required quantity has been drawn, the meter is again read and the difference between the registered quantity and the actual quantity noted. The sensibility of the meter is tested by running a small quantity of water through an orifice one-twentieth of an inch in diameter drilled in a second cap. Care should be taken

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* "The Care of a Water-meter." J. C. Whitney, Jour. N. E. Water-works Assoc'n, Vol. IX.

that during the tests no water is used from the service except through the faucet to which the caps are attached.

The cost of maintaining service-meters in works where a large proportion of the services are metered is about $1.00 per year per meter. Interest, sinking-, or depreciation-fund charges are not included in the above amount. This sum is divided approximately as follows:

Reading and inspection	$0.30	per meter per year.
Repairs	0.45	" " " "
Office expenses (computing and making bills, etc.)	0.25	" " " "
Total	$1.00	" " " "

In metered works the cost of reading meters is from 2 to 3 cents per meter for each reading. The annual cost of readings is, therefore, dependent upon the number of readings made during a year. The office expenses are affected by the number of bills rendered each consumer, and both expenses for reading, inspection, and clerical work are affected by the efficiency of the employés. The average cost of repairs per meter is dependent upon the number of meters in use and the service to which they have been subjected. Where meters are new or are set in comparatively large numbers, the average cost of repairs, obtained by dividing the total expenditure for repairs by the number of meters in service, is about 10 cents per meter per year or less. In works where a small proportion of the services are metered, and many of the meters are large, the average cost for repairs often exceeds $1.00 per year.

CHAPTER V.

CARE OF APPURTENANCES.

A REASONABLE amount of care should be exercised by the officials and employés in charge of a system of water-works to avoid any accident, due either to natural causes or neglect, which would impair the efficiency of the works or inconvenience consumers. Under favorable conditions the amount of care or oversight may be comparatively small, but usually considerable vigilance on the part of those in charge is required at times to maintain the system in good condition.

Intakes and screens should be kept clean and free from obstruction by ice, leaves, dead fish, or other débris. Ice, especially in the form of anchor- or needle-ice, is occasionally very troublesome and often interferes seriously with, and even at times entirely prevents, the flow of water into the intake-pipe. Anchor- and needle-ice form at the surface of the water, the crystals being drawn down to the intake, to which they adhere in large masses, by the movement of the water toward the latter when the water is agitated by wind, or when the intake is located in a flowing stream. These forms of ice do not occur when the surface is frozen, except occasionally in streams where the particles are carried under surface-ice from open water by the current. The tendency is for needle-ice to accumulate on the lee shore. Since this ice does not cause trouble in still water, platforms, rafts, or log-booms have been in instances moored over intakes as a means of protecting the latter. As methods for the removal or prevention of the formation of this ice in intake-pipes and on screens, which have been practiced with varying degrees of success, may be mentioned the use of poles, bars, and chains, steam, or hot water conveyed through pipes if the distance of

the intake from the shore or gate-house admits of this method, and the use of compressed air. The discharge of warm water from the condensing apparatus near the inlet, and the reversal of the flow in the intake-pipe have been found effective in some instances.

In a system supplied by pumps but not provided with reservoirs or stand-pipes having a comparatively large storage capacity it is important that the supply of water should be uninterrupted, and if funds are available reserve machinery should be installed in the pumping-station for emergency use. The interest charges on such additions to the plant will usually be offset by the saving due to the increased efficiency and decreased depreciation of the machinery in daily use, owing to the opportunity afforded for more frequent overhauling and repair of the latter. The friction of the engine and pump should be kept at a minimum by the use of suitable packing, properly placed, and good oils. Air in the pump, due to either leaks in the suction-pipe or an excess of air in the air-chamber, causes "pounding" and its absence in the air-chamber produces a like result. In the first instance, occupants of houses located at the higher elevations are annoyed by the presence of air in the service-pipes, and in the second the pulsations of the pump give rise to more or less water-hammer. If leaks in the suction-piping cannot be located and repaired the air may be removed by a vacuum-pump attached to a drum. If the discharge-chamber remains full of water, air should be forced in by a small pump until a sufficient "cushion" results.

Sand causes cutting and excessive wear of pump-plungers and rings, the water-cylinder linings of piston-pumps, and the valves, and should be prevented from entering the suction-pipe, or intercepted before it reaches the pump.

The amount of attention given to the boilers and engines is dependent in large measure upon the character of the employé in charge, and the position of engineer should be filled by an experienced and conscientious man. However, if everything runs smoothly little attention is often given to the slip of the pumps. Although the installation tests and duty trials of a pumping-engine show a slip of but from 1 to 3 per cent, and with good care this should not exceed the latter figure, instances

are on record where the amount of slip has been 40 and even 50 per cent of the nominal amount of water pumped. The slip may be due to short-stroking, loose water-pistons, worn valves, or a combination of these conditions, and tests should be made from time to time for its determination. In the absence of any measuring device for gaging the quantity of water delivered by the pump, an approximate test may be made by closing a valve in the discharge-main, care being taken that this shuts tight, and noting the number of strokes or revolutions made by the engine in a definite time both before and after closing the valve.

PUMP-SLIPPAGES IN A NUMBER OF CITIES AS DETERMINED BY PITOMETER MEASUREMENTS.

(Engineering News, Vol. LII, p. 538.)

Capacity million gallons daily.	City.	Description.	Per cent of pumpage.
10	Norfolk, Va.	High-duty, inside-packed.	4.2
4	" "	" "	11.9
1	So. Bend, Ind.	Power-driven, inside-packed.	57.0
1	" " "	" valves newly overhauled.	20.0
6	" " "	Duplex, inside-packed.	24.0
3	" " "	" "	19.0
1½	Oskaloosa, Ia.	Comp. ring and plunger	30.0
5	Columbus, O.	Ring and plunger, inc. jet condenser.	33.0
5	" "		33.0
5	Cleveland, O.	Inside-packed.	12.0
5	" "	"	10.1
10	New York.	Vertical triple, inside-packed (valves were adrift)	55.0
1	Brooklyn.	Well-pumping station.	25.0
10	Dayton, O.	New vertical triple, single-acting plungers, outside-packed.	1.5

The principal factor by which the comparative efficiency of the pumping-plant may be determined year by year is the cost of lifting a stated quantity of water a definite height. The units in general use for this purpose are 1,000,000 gallons and 1 foot respectively. As the cost of supplies and repairs is usually a small proportion of the annual expenditures for operation, and the salaries of the employés practically fixed in amount, the unit cost mentioned is dependent chiefly upon the amount and cost

of the coal consumed. Hence care and good judgment should be exercised in the purchase and use of the fuel-supply. The comparisons mentioned cannot be made unless sufficient data are recorded daily and preserved for reference in form similar to that outlined in Chapter I.

The unit cost of pumping varies within wide limits and is affected by many conditions, as size and efficiency of plant, quantity pumped, total head, average daily duration of pumping, cost of coal, character of labor, etc., similar conditions rarely obtaining in any two localities, but the statistics given in the following table may be found to be of interest.

COST OF PUMPING IN VARIOUS LOCALITIES.

(From Jour. N. E. Water-works Assoc'n, Vol. XIX, 1905.)

City or town.	Total pumpage for year in millions of gallons.	Average dynamic head in feet.	Duty in million foot-pounds per 100 lbs. of coal.	Cost per million gallons raised one foot.
Oberlin, Ohio	73	84	11	$0.35
Bay City, Mich.	1,091	113	21	0.071
Reading, Mass.	56	240	24	0.269
Middleboro, Mass.	93	204	25	0.14
Billerica, Mass.	37	316	30	0.171
Atlantic City, N. J.	1,627	126	31	0.087
Westerly, R. I.	251	210	31	0.108
Ware, Mass.	128	244	40	0.100
Marlboro, Mass.	221	173	43	0.077
Newton, Mass.	800	257	47	0.05
Woburn, Mass.	520	214	47	0.074
Woonsocket, R. I.	400	240	48	0.057
Cleveland, Ohio	22,535	219	61	0.025
Lawrence, Mass.	78	318	86	0.096
	949	185	70	0.071
Waltham, Mass.	759	180	97	0.059
Cambridge, Mass.	2831	194	131	0.031
Metropolitan Water-works, Mass.	576	121	65	0.059
	293	128	121	0.035
	10,523	129	135	0.022
	20,269	55	110	0.030
	2,927	129	132	0.031

Stand-pipes should be examined at intervals, leaks in joints remedied by calking, and bolts or rivets which have been loosened by the movement of ice in the interior of the tank or from other causes tightened. The interior of the stand-pipe should be painted at intervals of two or three years, or as local conditions may require,

a durable paint of good quality being used for that purpose. The metal should be clean and free from loose scale before the paint is applied. Cleaning by aid of a sand-blast is the most effective method of treatment, but is usually not resorted to on account of the cost. The scale and rust may be removed by steel scrapers—articles made from flat files or old saw-blades ground on one edge and set in a suitable handle have given satisfaction—after which the metal should be thoroughly brushed with steel brushes. The exterior of the stand-pipe does not usually require repainting as often as the interior, and paint of a somewhat cheaper grade may be used.

All public fire-hydrants connected with the works should be inspected at least twice yearly: once in the autumn before cold weather sets in, and again in the spring. At these inspections the hydrants should be oiled and the threads of the nozzles and caps lubricated. In cold climates the fall inspection is of the utmost importance, as the value of the system for fire-protection depends upon the condition of the hydrants, and considerable loss may be incurred if these appliances are out of order or frozen. A hydrant may be rendered temporarily useless in cold weather by the freezing of water in the barrel. The water may come from the mains through a leaky valve, or surface water may find its way into the space between a hydrant and the frost-case and enter the drip, or water may remain in a hydrant after use owing to an obstructed drip. In the first instance the leakage may be due to a cut or worn valve, or a sprung rod, and when such leakage is apparent the defective part should be repaired or renewed at once. In the second instance the drip should be plugged and the hydrant pumped dry after use. A hydrant in which the drip is obstructed by tree-roots, rust, pebbles, or other matter will drain very slowly, if at all, and the fall of the water in the barrel when the hydrant is closed should be observed at the time of inspection. Tree-roots are very troublesome, as the cutting away of the roots merely postpones the time when the drip will be again closed until a new growth is made. The better way to deal with cases of this nature is to connect the drip with the street sewer by a service-pipe, the street surface-water drains usually being laid above the level of the bottom of the hydrant. Accumulations of

rust and earth in the drip may be removed by the admission of water to the hydrant after the nozzle-caps have been made air-tight, after which the main valve is closed, when the escape of the water under the pressure due to the compressed air in the top of the barrel scours the waste-opening clean.

The consensus of opinion among water-works superintendents is that the main valve of a hydrant should not be opened between the fall and spring inspections except in case of fire. In very cold weather the hydrants should be examined at intervals for the presence of ice or water in the barrel above the valve or drip by

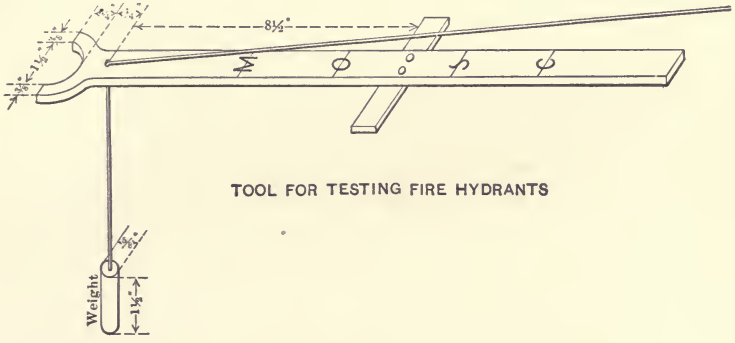

FIG. 82.

the use of a flexible rod—an old whip answering this purpose—or a metal ball attached to a cord. The device shown in Fig. 82* has been used with success at Reading, Pa., for this purpose. A knot or marker on the cord which supports the weight indicates the depth of the barrel, the different depths of the several makes of hydrants being indicated by marks on the handle.

Doubtful hydrants may be given a dose of wood-alcohol or salt, but the continuous use of the latter is objectionable owing to its corrosive action.

Hydrants which have been used for fire or other purposes during the winter months should be examined after such use.

Nozzle-caps should never be left loose, as mischievous boys often take advantage of the opportunity thus offered and place

* E. L. Nuebling, "A Simple Device for Testing Fire-hydrants," Proceedings Am. Water-works Assoc'n, 1903.

stones and sticks in the barrel. These may be washed out through the steamer-nozzle, or the top of the hydrant if the cap is removed, or if these methods fail, tongs or hooks may be used to remove the obstructions.

The indiscriminate use of hydrants by employés of street or sewer departments, contractors, or private individuals, or for the filling of street-sprinkling carts should never be permitted. In the former cases the hydrant should be opened and closed by an employé of the water department, or if the water is to be used for some time an ordinary wheel-valve, with or without a reducing coupling, should be placed on one nozzle and the hydrant opened in the morning and closed at night by a competent man. Sprinkling-carts should be filled from stand-pipes or water-cranes provided for that purpose.

The nuts securing the valves of plug-hydrants to the rod should not be allowed to become loose, or if the valve is not held firm, washers should be added, as water-hammer will result from the closing of such hydrants if the valve seats with the pressure.

Valves and hydrants should be operated at a slow rate in order that water-hammer may not be produced. When water is turned on a main which has been drained for any purpose, one or more hydrants located at dead ends and summits should be opened to allow the air contained in the pipe-line to escape. Air should be admitted to the interior of a long pipe-line near the controlling valve when the pipe is drained in cases where the valve is located at a higher elevation than the outlet. Disregard of this precaution has at times resulted in injury to or destruction of the valve, or of portions of the main below. This effect has been accounted for by the theory that, as a result of the vacuum produced in the main between the valve and the receding column of water, a portion of the latter is detached and returns with considerable momentum, the force of impact being sufficient to break the valve or main.*

The valves of a system should be examined occasionally, and leaking stuffing-boxes made tight by a renewal of the packing about the valve-stem. Packing lubricated with oil and plum-

* Jour. N. E. Water-works Assoc'n, Vol. XVI, p. 136.

bago is suitable for this purpose. Care should be taken in the operation of a valve that the disc or discs are not wedged, as stripping of the threads or breaking of the valve-stem is easily accomplished in this case. After opening, a valve should not be left with the wedge set tight, but the gate-wrench should be turned in a reverse direction a few times in order that the disc may be left loose.

Valves on circuit lines should be examined frequently, especially after they have been closed for repair or other work, as such valves are often left closed by careless workmen, no indication being given that such is the case, as would result when the line is controlled by a single valve.

The tops of street- and service-boxes should be maintained flush with the surface of the ground or pavement. If the boxes are lifted by frost action, they may, if of the telescopic pattern, be driven down into place with a wooden maul. Hot water poured into the box will soften the ground and prove of service in obstinate cases.

As is the case with hydrants, valve-boxes are often made receptacles for sticks, stones, and other rubbish when located in unfrequented places and alleys. Tongs, hooks, scoops, and other contrivances may be used to clear the box in these cases. A circular mirror about 3 inches in diameter will be found very useful in locating obstructions in the box, as the sunlight can be reflected into the box by the mirror, thus lighting the interior. The mirror will also be found useful in locating the tee head of a curb-cock, and by its use the wrench may be placed directly on the handle of the shut-off. Without the mirror more or less time is lost in fitting the wrench in place, and often the shut-off is struck and turned to one side.

The space between the valve-box and its cover is often filled with ice, and the entire cover at times is covered by this substance. Benzine or gasolene placed in a cup-shaped depression cut in the ice over the cover will on ignition melt the ice and loosen the cover. A handful of waste soaked with kerosene will accomplish a like end. When but little snow is upon the surface of the street, frequent applications of salt to the covers will tend to keep them clear.

The flow of water through service connections is often obstructed, entirely or in part, by the presence of small fish or eels in the corporation cock or service-pipe. If the fish or eel has not passed the corporation cock, the service-pipe may be disconnected in the basement of the building at the shut-off, a small force-pump attached, and sufficient pressure applied to force the obstacle out into the main. At the same time a hydrant or blow-off located on the main beyond the service in question should be opened. Obstructions of this nature between the corporation cock and the building may be removed by inserting a stiff wire, looped at the forward end, into the service-pipe from the basement. The material is broken up by the wire and brought out by the water. A wire of this kind will also be found useful in cleaning pipes which are clogged with rust. The looped wire should be pushed into the service-pipe against the pressure and the material loosened. The detached particles, and masses of rust, will be washed out by the running water, and the service-pipes should not be connected up again until the water runs clear, otherwise the fragments will lodge in faucets or ball-cocks and cause trouble.

The closing of the inlet of the corporation cock as the result of tuberculation of the main at this point may be remedied by excavating to the former, disconnecting the gooseneck, and drilling out the opening. If the corporation cock, when first placed, projects into the main one-half to three-quarters of an inch, little trouble should be experienced from obstructions due to the above cause. The decrease in the flow of water due to obstruction of the connection by rust will, unlike that due to the presence of eels or fish, usually be very gradual.

More or less sediment, even with apparently clear waters, is deposited in the main pipes of a system, and the capacity of the mains is often decreased by the formation of growths or incrustations upon the interior. The sediment should be removed by increasing the velocity of flow through the pipes, and this may be accomplished by the discharge of water from fire-hydrants and blow-offs. When a main is shut off and drained for any purpose the water which first enters when it is again placed in

service should be wasted in this manner, or complaints that the water is dirty will be received from consumers.

The presence of incrustations upon the interior of cast-iron pipes, or of growths of pipe-moss or fresh-water sponge upon pipes and conduits causes a reduction in the delivering capacity of the main affected by such growths. This reduction is due to the contraction of area of the pipe or conduit, to the retardation caused by the increased frictional resistance offered to the flow, and to the production of eddies. The formation of incrustations and growths is dependent upon the character of the water and the condition of the interior surface of the pipe. Incrustation does not occur with some waters, but in other cases the coatings, as at present applied to commercial pipe, do not entirely prevent the formation of tubercules. The tubercules vary in size and appearance, the smaller presenting an appearance not unlike a blister, the larger resembling cones of varied form. These are usually hollow or contain a soft spongy mass of rust and other matter, and when first removed from a pipe the outside is quite hard, but after exposure to the air for some time the mass often crumbles and may be easily crushed by the hand.

Tubercules or other growths may be removed by scrapers or brushes drawn through the pipes by hand or forced through by water-pressure. Several devices have been used with success for this purpose in Great Britain and its dependencies, but, with the exception of a few experimental trials, the method of scraping has not been introduced in any water-works systems in the United States. The scrapers used consist of a central shaft to which are attached steel cutting-blades or knives, and pistons of rubber, leather, wood, metal, or a combination of these materials. The shaft and cutting-blades are usually flexible, or the latter are held in place against the pipe by springs, in order that the apparatus may pass slight obstructions which cannot be removed, and around curves of thirty or more feet in radius. The hand-scrapers are little used except for cleaning short sections, as their use necessitates frequent cutting of the main. The automatic scrapers are introduced into the pipe through openings cut in the latter, or hatch-boxes designed for the purpose, and when in place are forced through the main by the pressure of the water behind them,

the course of the apparatus through the pipe being noted by observers on the surface, as the noise due to the action of the device is noticeable above ground.

The results obtained by the use of these scrapers have been very satisfactory as regards the increase in the discharge of the pipes cleaned, since the delivering capacity of the main so treated has been increased from 25 to 100 per cent in several instances. This increase in capacity is, however, not permanent, as the incrustations removed are soon replaced by a new growth. In order that the method may be of value the pipes must be scraped at frequent intervals, and hatch-boxes are set permanently in place at convenient points to facilitate the operation.

The cost of the work is dependent upon the age of the main and the amount and character of the incrustation within it, the cost of rescraping being usually much less than the first cost of cleaning.

The original cost of cleaning 4.3 miles of 24-inch supply-main at St. John, N. B., in 1897–98 was about $64.00 per mile, or 12 cents per foot. The scraper used (Fig. 83) cost about $40.00 and a distance of about 94.5 miles was covered to accomplish the work.*

In 1886–87 58,000 feet of 6-inch and 20,300 feet of 12-inch distribution mains were cleaned at Boston, Mass., at a cost of 14 cents and 20.6 cents per foot respectively, the costs including a royalty payment of 5 cents per foot for the right to use the scraper.†

The supply-mains at Torquay, England, are scraped every year at a cost of about $3.50 per mile for labor. The mains are 9 and 10 inches in diameter and the scraper used cost about $112.00.‡

At Melbourne, Australia, the first cost of cleaning about 164 miles of pipe from 4 to 12 inches in diameter from 1897 to 1902 varied from about $10.00 to $120.00 per mile of main cleaned.

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* "Cleaning a Water-main in St. John, N. B." Wm. Murdoch, Jour. N. E. Water-works Assoc'n, Vol. XIII.

† Discussion of above paper by Dexter Brackett.

‡ "English Practice in Cleaning Water-mains with Scrapers and Brushes," Eng. News, Vol. XLIV, p. 154.

FIG. 83.—Pipe-scraper.

The scrapers used cost from about $60.00 for the 4-inch size to $325.00 for the 12-inch size.*

At Halifax, N. S., in 1898, 112,803 feet of water-pipe from 3 to 24 inches in diameter were recleaned at an average cost of about $20.00 per mile.†

The above method of increasing the delivery of pipes should be found of value in the case of small cast-iron supply-mains, the capacity of which has been impaired by the presence of incrustations.

The flow of water through supply- or force-mains which are not tapped for service connections or hydrants is at times ob-

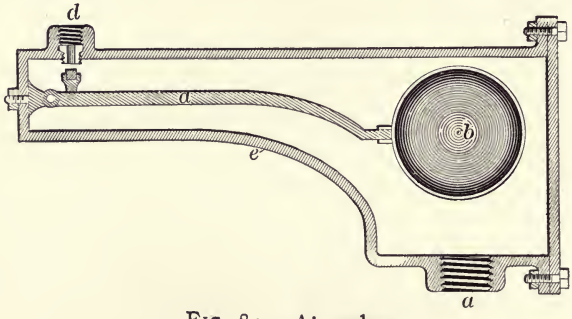

FIG. 84.—Air-valve.

structed by accumulations of air at high points or summits along the main, and automatic air-valves are used to allow such accumulations to escape from the pipes. These devices usually consist of a chamber which is attached to the main at summits, and which contains a ball or float. An accumulation of air in the chamber displaces the water, and the consequent downward movement of the float opens the valve, the air escapes, and the water re-entering the chamber lifts the float, thereby closing the valve. Types of automatic air-valves are shown in Figs. 84 and 85.

Mains which are exposed to the weather, as is the case with pipes laid over streams or connected with elevated tanks, or which are laid with an insufficient covering of earth or other material, are liable to freeze in cold weather unless the circulation or flow of

* "Automatic Water-pipe Scrapers at Melbourne, Australia," Eng. News, Vol. LIII, p. 14.

† "Experience with Mechanical Scrapers for Cleaning Water-mains at Halifax, N. S.," Eng. News, Vol. XLIV, p. 127.

water within the pipes is continuous. As measures of precaution against this liability, pipes are surrounded with hair-felt, mineral wool, asbestos, wool waste, or leather cuttings; or the pipes are boxed with saw-dust, charcoal, rosin, tanbark, asphalt, or tar-concrete; or dependence is placed upon air which is confined by double or single boxing, in which case a space of from 2 to 6 inches is left between the pipe and the sides of the box.

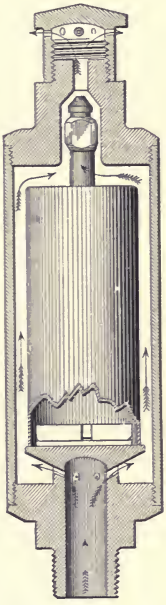

FIG. 85.—Air-valve.

The use of any fibrous material will prove ineffectual if the material becomes wet, as it will then freeze and lose its non-conducting properties. The method of boxing has been the most satisfactory in many cases, provided there is no opportunity for the air within the box to circulate. In extreme cases the confined air has been warmed by steam or hot-water pipes enclosed within the box and supplied from a convenient boiler or heater.

When service-pipes are laid where ledge is encountered, and the property-owner does not desire to incur the expense necessary to place the pipe at the proper depth, the pipe may be laid in a somewhat shallow trench and thoroughly bedded in asphalt or tar-concrete similar in composition to that used for walks.

Mains laid in or under bridges should be provided with taps or blow-offs through which the water may be drawn off and the pipe drained in case it becomes necessary to shut off the main during cold weather.

Notwithstanding precautions taken to prevent freezing, mains, hydrants, and service connections, particularly the latter, occasionally freeze. When the flow of water within a main is stopped by the formation of ice, no time should be lost, but measures for the restoration of the flow should be at once undertaken, otherwise the formation of ice, which commences at an exposed point, will often continue for some distance along the main. If the pipe is exposed the careful application of steam or hot

water will heat the metal and thaw the ice; or the pipe may be surrounded with unslaked lime, which is moistened and covered in order that the heat produced may be utilized. The application of fire to a frozen pipe or hydrant is a questionable proceeding, as the cast-iron often cracks under this treatment.

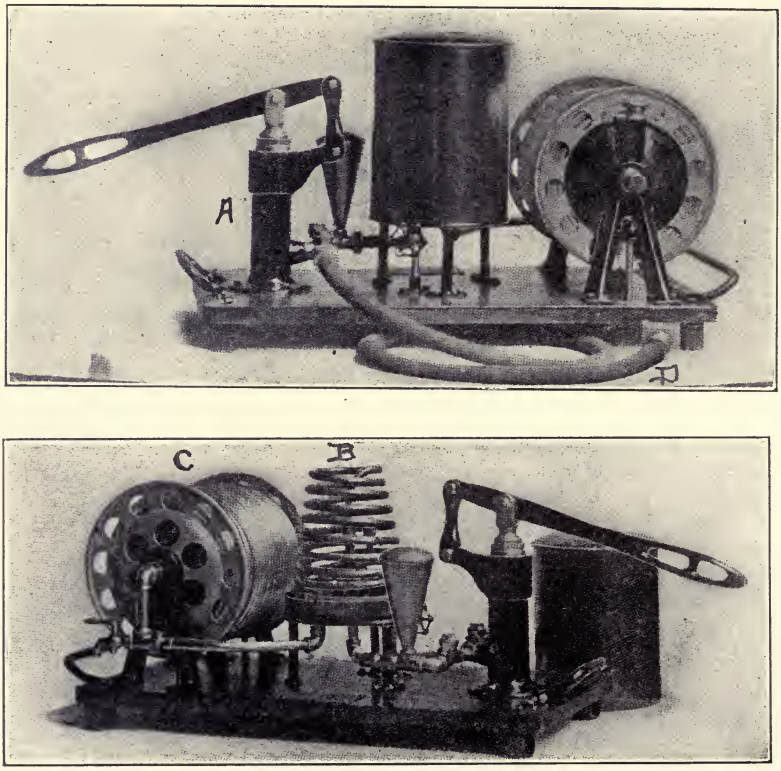

FIG. 86.—The Burbank Thawing Machine.

A, Force-pump; *B*, Coal Boiler and Gasolene Burner; *C*, Block-tin Pipe on Reel; *D*, Suction Hose.

Frozen hydrants may be thawed by steam generated in a small portable boiler and introduced into the hydrant through a nozzle by a short hose, or hot water may be pumped into the hydrant from a pail heated over an ordinary plumbers' furnace. After a hydrant is thawed, it should be pumped dry and examined at frequent intervals.

Service-pipes may be thawed in like manner by the use of a block-tin pipe of $\frac{3}{16}$ or $\frac{1}{4}$ inch internal diameter, which is inserted into the pipe at the point where the latter enters the building supplied, or at the stop and waste cock. The steam used is generated or the water heated in the manner mentioned, one man attending to the boiler or heater and a second forcing the thaw-pipe into the connection. The block-tin pipe is quite flexible and will pass easy bends but not an elbow. If the ground surrounding the service connection is frozen the pipe will freeze again unless the water is allowed to run.

To obviate the necessity of allowing the water to flow continuously through a service connection which has been thawed, the following method of thawing is used at Holyoke, Mass. Steam under 50 to 60 pounds pressure from a portable boiler is used through hose, to the end of which is attached a piece of $\frac{3}{4}$-inch iron pipe about 6 feet long, provided with handles. With steam on, the pipe is easily forced down through the ground to the service-pipe, where it is allowed to remain a short time. This operation is repeated at points along the line of the connection until the pipe is entirely thawed. It has been the experience at Holyoke that services thawed in this manner do not again freeze during the season.

Short lengths of main may be thawed by the use of steam or hot water used in a manner similar to that in the case of services, but larger boilers or heaters, and larger hose or thaw-pipes, are required in such cases.

In localities where the electric current is available it may be advantageously used for thawing frozen pipes, the heat produced by the passage of a current of sufficient strength through the pipes being sufficient to melt the contained ice. This method was made practicable by Professors Jackson and Wood of the University of Wisconsin in 1898–99, and is outlined in a circular issued by that institution, as follows:

"The current which is required for thawing service-pipes satisfactorily is from 200 to 300 amperes. The source of current should have a pressure of not less than 50 volts. Where electric-light lines carrying alternating currents are available, a transformer or transformers in parallel may be used as a source of current.

It is very important that direct connection of pipes to house lines be avoided on account of danger of fire in which the house is placed by such connection. Where alternating currents are not available, continuous-current feeder-lines may be used, but these should be entirely separated from the distributing network of conductors.

"The accompanying sketch will show the way in which the appliances should be connected when an alternating current is

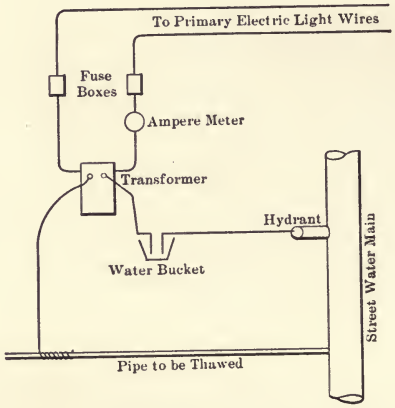

FIG. 87.—Thawing Services by the Electric Current.

used with transformer. The secondary leads from the transformer should be quite large, such as No. 3 B. & S. gauge or larger. In making connection to the pipes, one of the secondary leads should be taken into the house to which the frozen service-pipe leads and contact made at that point by some form of metallic clamp or by simply giving the conductor two or three tight twists about the pipe at any point where the pipe is exposed or at a faucet in the house. The other secondary lead should be put in contact with the water system outside of the house, and in a similar manner. This contact may be made at a hydrant, or at an adjoining service-box, or at pipes in a neighboring house. When there are two houses near together, each with frozen service-pipes, the two secondary leads may be connected to the pipes within these houses and both frozen service-pipes thawed out at once.

"While the thawing process is going on, the faucet should be open in the house to which the service-pipe leads. In one of the

secondary leads should be inserted a water resistance, which consists (for convenience) of a bucket of water containing a bowlful of salt, and two sheet-iron or copper plates to which the ends of the severed lead are attached. This serves to control the current. In the primary leads from the electric-light line to the transformer, it is highly desirable to have a fuse in each lead, and an ampere-meter. When all connections are made, the plates are placed in the bucket and are then moved towards each other until the ampere-meter records a proper current. If the primary pressure is 1000 volts and the secondary pressure 50 volts, the current should ordinarily approach 15 amperes. If the primary pressure is 2000 volts and the secondary pressure 50 volts, the ampere-meter reading should ordinarily approach 7.5 amperes.

"Water ordinarily begins to flow in a time not much less than ten minutes or not greater than one hour. If the secondary current is quite close to 300 amperes the period seldom exceeds one-half hour. The frozen pipes are often split by the action of the frozen water, and these at once begin to leak when the ice is thawed away. For this reason it is desirable to have a plumber where he may be readily called to care for the leaky pipe.

"The electric current when properly used will not damage the pipes. It is desirable to watch brass and iron connections to lead or iron service-pipes, as they sometimes heat on account of poor contact. If such heating appears to be excessive, the current may be reduced with a resulting increase in the duration of time for thawing.

"After the pipe has been thawed it is desirable to let the water run continuously for a considerable time, inasmuch as the ground all around the pipe is frozen and the pipe is liable to freeze up again unless the water circulates."

This method of thawing frozen water-pipes is now used with success in a number of works, and the cost of thawing has been found to be about $4.00 to $7.00 per service thawed. The method is applicable in the case of frozen mains, but the current and voltage and the time required are increased. Several manufacturers of electrical apparatus have recently placed on the market portable outfits designed for this particular purpose.

CHAPTER VI.

ALTERATIONS AND REPAIRS.

CONTINGENCIES frequently arise, even in well-designed works, that necessitate alterations in existing structures before additions can be made thereto. Such events more commonly occur in connection with the distribution system. Provision must often be

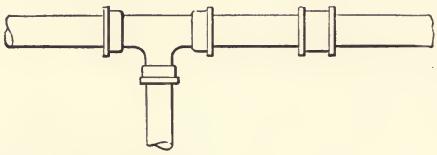

FIG. 88.—Cutting-in a Branch.

made for branch pipes to serve new streets, large consumers, or additional hydrants. Connections with existing mains for such purposes are made in several ways. The method commonly used in the case of small and relatively unimportant mains is to cut out a short section of the pipe and insert in the line a suitable special casting. A short piece of pipe and a sleeve are used to close the opening made in the main.

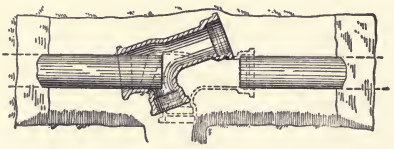

FIG. 89.—Dunham Cutting-in Special.

A saving in labor and material may be effected by the use of a "cutting-in" special in the above instance. These specials are so constructed that one bell end can be slipped over one of the

ends of the severed pipe and the casting placed in position, as shown in Fig. 89.

Connections with large mains may be made by the use of split branch sleeves. The two sections of a sleeve of this kind are held in place by bolts, an opening of the required size cut through the pipe wall, and the space between the ends of the sleeve and the pipe filled with lead and calked.

Bolted sleeves provided with a boss, which is tapped to receive a service connection, may be used in a like manner in place of the tapping-band mentioned in Chapter III.

The disadvantage of the above methods is that the main pipe to which the connection is made must be shut off and drained

FIG. 90.—Split Branch-sleeve.

while the work is performed. This procedure inconveniences consumers, often to a considerable extent, and the shutting off of a large and important main may at times be avoided by the use of a special tapping-machine. This machine operates in a manner somewhat similar to that of a service tapping-machine, and by its use the sleeve is placed, the main tapped, and a valve set without discontinuing the supply. Since these machines are somewhat expensive, they are rarely owned by small works, but are usually to be found in all large cities and may be at times rented.

The cutting of a pipe in the trench by hand with a chisel and hammer is a slow and laborious process, and the pipe is often cracked or broken when cut in this manner. Several machines have been designed for this purpose and may be purchased from dealers in water-works supplies.

It is practically impossible to separate two lengths of water-pipe without injuring one of the pipes, when there is a bead on the spigot end of one and the joint has been well calked, except

by melting or cutting out a large part of the lead in the joint. As a general rule, the method of cutting by hand should be used only as a last resort, as it is very difficult and tedious work to

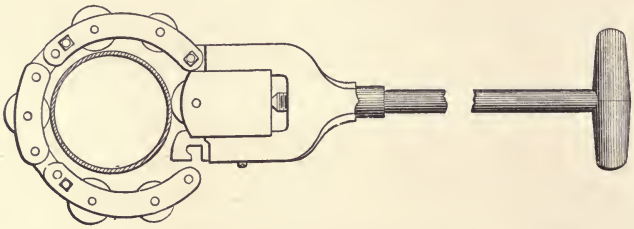

FIG. 91.—Hall Pipe-cutter.

remove lead from a joint in this manner. A wood fire may be built about the joint and the lead slowly melted, but a gasolene- or kerosene-flame under pressure is much more efficient. A No. 3 Wells light was used successfully at Providence, R. I., in 1898 for melting out the lead joints in a line of 24-inch cast-iron

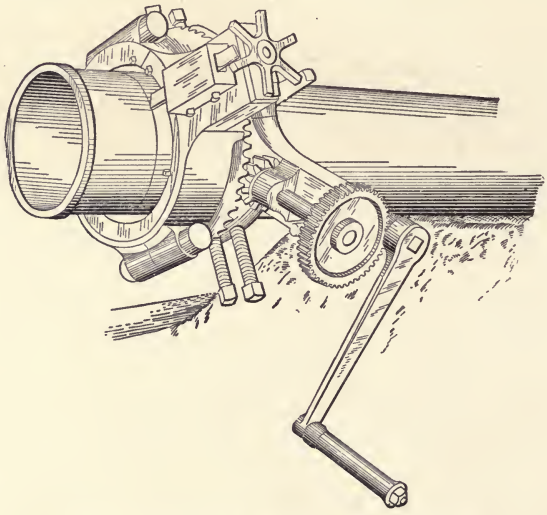

FIG. 92.—French Pipe-cutting Machine.

pipe. A sheet-iron hood was used to concentrate the heat from the burner upon the joint. Kerosene-oil was used under a pressure of about 25 pounds to the square inch, and about two gal-

lons were consumed for each joint melted. The time required to melt out a joint was about one hour.

An apparatus for melting the lead in the joints of small pipe-lines may be made with two coils similar to those in an ordinary plumbers' furnace, which are attached to a piece of iron pipe. This serves as a handle and is connected by rubber tubing with a small tank partially filled with gasolene. The air in the tank

FIG. 93.—Method of Melting Out Lead Joints of Water-pipe. (Providence Reports, 1898.)

is compressed to the desired degree by a small force-pump and the flame from the burners directed against the joint.

The re-establishment of street grades at times necessitates the re-location of the grade of water-pipes, and when supply-mains are affected by such changes, the pipes must often be moved while under pressure. Where the ground under the pipe is firm and compact, the line may be lowered by excavating the earth from beneath the pipes, leaving each length supported by a short section of unexcavated material or by blocking. The line is then carefully lowered by removing the material supporting the pipe or by digging from beneath the blocking. The

trench should be excavated of sufficient width to allow room for work and bracing should be placed between the pipe-line and the side of the trench to prevent any lateral movement of the main. Planks or timbers held in place by iron screw-braces are used for this purpose. These extension screw-braces are also very convenient in bracing the sides of excavations during repairs.

Where excavations for sewers or other purposes are made beneath water-pipes these pipes should be supported in such manner that the line may be held in place if settlement of the ground on either side of the excavation takes place while the work is in progress. Water-pipes are usually supported from timbers placed across the excavation, the main being held by ropes or chains. The supporting ropes and chains are kept tight, and the water-pipe maintained at grade, by inserting a pick-handle or iron bar between the supporting lines and twisting these whenever the ropes or chains become slack, or when settlement of the supporting timbers tends to lower the pipe.

A more satisfactory method at times is to support the pipe from a vertical iron bar which is provided with threads which engage those of a heavy plate upheld by cross-timbers. This form of apparatus is also of service in raising or lowering mains where the ground is unstable.

The excavation of frozen material is at best expensive, particularly when picks and wedges only are used. The building of wood fires along the line of the trench results in the thawing of but a few inches of the surface and the adoption of this method is not usually advisable. Better results may be obtained by the use of unslaked lime, which is placed in a layer along the trench, moistened, and covered with manure or tight boxes. The thawing may be accomplished economically by the use of steam from a portable boiler if one be available. At Providence, R. I.,* ground which was frozen to a depth of from 12 to 24 inches was thawed by this method at a cost of about 1.7 cents per square foot. Boxes 72 feet long, 20 inches wide at the top, and 24 inches wide at the bottom were placed on

* Jour. N. E. Water-works Assoc'n, Vol. XIII, pp. 90, 91.

the line of the trench, and after all cracks and openings were closed with earth or bagging, the boxes were filled with steam under about 50 pounds pressure from a small portable boiler. The steam was introduced at one end of the box and a small opening was left at the opposite end to cause circulation. The boxes were built of spruce boards and were made in sections 12 feet long.

Water from leaks in main pipes or service connections usually appears upon the surface above the point at which the water is escaping, unless the street pavement is water-tight or the

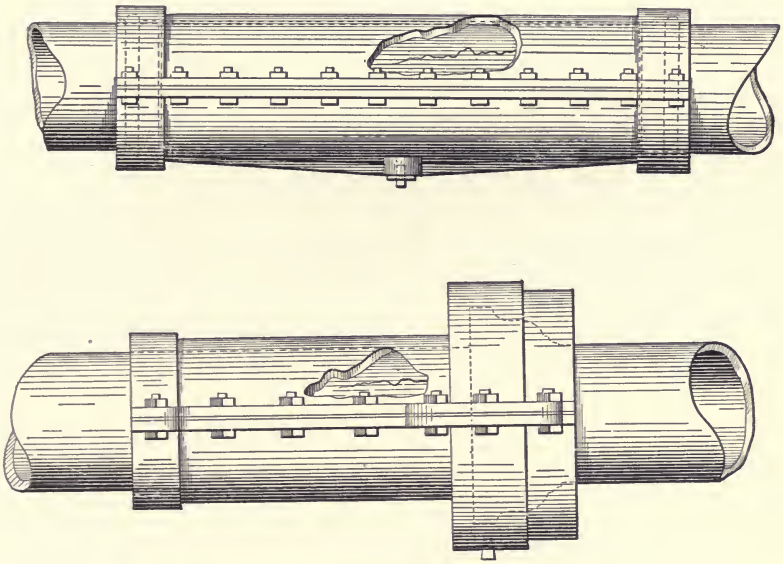

FIG. 94.—Kellogg Repair Sleeves.

ground frozen. In the latter instances the water will often appear outside the limits of the pavement, or in grass ground in which the frost has penetrated to a comparatively slight depth. The appearance on the surface will be usually at a point about opposite the leak. The water often, however, in the above instances follows trenches in which water or sewer connections are laid, and appears at the surface in the immediate vicinity of valve-boxes or hydrants, or finds its way into the basements or cellars of adjacent buildings.

An exception to the general rule is found at times when the

leak is in close proximity to a culvert or a defective sewer. The presence of a leak may then not be apparent at the surface.

Leaks in cast-iron water-pipes are, in general, due to defective joints, cracks, or blow-holes in the pipe, or pittings caused by electrolysis. Leakage at a joint may be usually remedied by re-calking the lead. Pipes are occasionally laid with slight fractures at the spigot ends or a pipe may be partially split by the freezing of the water contained therein. If the crack is of considerable length, the damaged pipe is usually broken out and

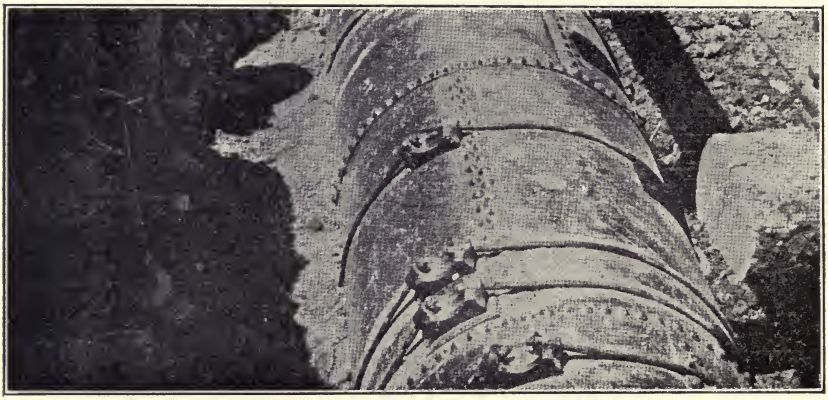

FIG. 95.—Method of Repairing Steel Main, Cambridge, Mass.

replaced with a new piece. If the crack is short, a split sleeve may be bolted about the pipe and the space between pipe and sleeve filled with lead.

Cracked pipes may be repaired with the patent sleeves shown in Fig. 94 without shutting off the water from the main. The sleeve is placed in position with a gasket of asbestos between the two sections, the joints at the ends leaded and calked. The water from the leak is allowed to escape through a hole provided for that purpose, in the lower section of the sleeve, while the sleeve is placed and secured in position, after which the opening is closed with a screw-plug.

Leakage through blow-holes in the pipe may be stopped by tapping out the hole and inserting a brass screw-plug, or a short bolted sleeve may be used.

Small leaks frequently occur in riveted steel conduits and

these are repaired whenever possible without discontinuing the supply. Small holes in the steel conduit of the Rochester, N. Y., water-works are closed with a wooden plug. A piece of sheet lead somewhat larger than the hole is then placed on the pipe and covered with a steel patch, which is held in place by heavy iron clamps or bands encircling the main. The outside edges of the lead patch are calked. A somewhat similar method is used in Cambridge, Mass., for repairing the steel mains. Rubber packing one-quarter of an inch in thickness is placed over the hole and covered with a steel patch five-sixteenths of an inch thick rolled to fit the outside of the pipe. The patch is held in place by shoes and bands as shown in Fig. 95.

Some difficulty is at times experienced in running lead joints when water is present in the pipe-bell, as the steam generated by the heat from the lead often loosens the pipe-jointer, thus permitting the lead to escape, or blows out some of the metal. When the water comes from the interior of the pipe-line, the flow may be checked by calking a piece of small lead pipe into the joint before yarning; or yarn smeared with lard or tallow may be used in making the joint. If but a small quantity of water is flowing, oil poured into the joint immediately before the lead will be found beneficial.

Leaks in submerged pipe-lines are located by shutting off the line and pumping in air. The character of the leak is usually ascertained, and the repairs made, by a diver. When split sleeves are placed on submerged pipes, the sleeve may be fitted to a pipe similar to that needing repair and the lead joint poured but not calked. The joint is then cut in two, the separate portions of the joint and sleeve placed, the bolts tightened, and the lead calked by a diver.

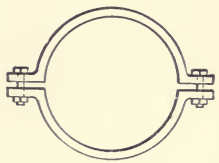

FIG. 96.—Clamp.

The particular size or pattern of split sleeve needed to repair a damaged pipe is not always on hand when wanted, and temporary repairs may be made by winding rubber sheeting or painted canvas about the pipe, this being held firmly in place by a number of turns of small rope or by wire; or a piece of rubber gasket may be held tightly against the pipe by a thin plate of wrought iron

hammered to fit the surface of the main and secured by suitable clamps or bands. An emergency clamp may be made of a piece of strap-iron of a length somewhat less than the circumference of the pipe, to the ends of which angle-irons are riveted. The angle-irons are drilled for the reception of bolts, by which the clamp is held in place.

The common causes of leaks in service connections are loose-fitting corporation cocks and imperfect joints between the pipe and the stop and waste cocks, or between adjoining lengths of pipe. It is usually necessary in such instances to shut the water off at either the corporation or curb cock and tighten the defective joint. Defective cup joints in lead pipe should be melted out and remade. The threads of iron pipe on the street side of the house stop and waste cock frequently rust away, when it is necessary to cut new threads and replace the stop-cock on the pipe. When shut-off cocks are not placed at the curb, the water cannot be shut off without digging to the corporation cock. Frequently the curb cock is not properly located and cannot be found when wanted without digging. Several devices have been made for use under these conditions to check the flow of water while making repairs without recourse to excavation in the street. These devices are very similar in operation to the ordinary testing-plug used by plumbers, the pipe being closed by a rubber ring which is compressed between two metallic discs and expanded against the interior of the pipe. One pattern is so constructed that the

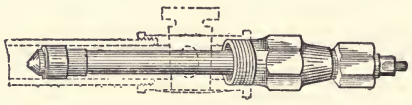

FIG. 97.—Thomas-Nusser Stop-cock Replacer

apparatus may be attached to the house side of the stop-cock, the cock opened, the plug inserted a sufficient distance in the pipe, and the stop-cock removed, together with a portion of the apparatus. After the new shut-off has been placed, or new threads cut and the old one replaced, the plug may be removed by reversing the method of procedure. No water escapes under pressure during the operation.

Service connections must at times be laid beneath steam- or electric-railway tracks, or paved walks or driveways, and at such times the inconvenience and expense incidental to the work may be reduced by the use of machines designed to force pipe through the soil, provided the conditions are favorable to their operation. These machines may also be used to advantage for renewing service connections under like conditions, or where excavations in lawns are undesirable. In these instances the ends of the service connection and the pipe at the curb cock are made accessible, the old pipe withdrawn and the new placed by attaching it to one end of the former.

Leaks in earthen dams or reservoir embankments should receive prompt attention if the escaping water is washing any material out of the structure. Usually the water must be drawn down and permanent repairs made on the water side of the embankment. When located, the hole in the bank is enlarged and the opening carefully filled with well-compacted puddle. At times it may be necessary to excavate from the top of the bank to a point slightly below the leak and refill the excavation with selected material.

Leaks on the water side of masonry dams, or on the inside of conduits, are often located by throwing sawdust into the water and noting the points where the sawdust is drawn into the masonry after the water has been drawn down and the surface exposed. Small leaks are occasionally stopped in this manner. Large apertures should be cleaned and filled with a grout of Portland cement. Cracks may be repaired by calking with soft ribbon lead.

The masonry linings of distribution reservoirs sometimes crack in consequence of settlement of the underlying material, frost action, or from other causes. Coats of plaster upon stone paving or concrete, if improperly applied, are liable to scale off in course of time, to the detriment of the appearance and water-tightness of the reservoir.

Leakage from a reservoir due to the above causes is remedied by repairing the lining or by replacing it with new material. Cracks or open joints in an otherwise impervious lining are repaired by filling the openings with asphalt or Portland-cement mortar. Old linings are made more or less water-tight by the application

of asphalt, Portland-cement grout, washes containing certain chemicals, or an additional lining of concrete.

Asphalt was used in repairing the Queen Lane reservoir of the Philadelphia water-works in the following manner: "The material used was composed of 4 parts of Bermudez asphalt and 1 part of the liquid or F grade of Alcatraz asphalt. These were melted together and poured on the surface to be treated, which had been previously primed with a thin coating consisting of 3 parts of the F grade of Alcatraz asphalt and 7 parts of gasolene. The asphalt was only partly liquid, and it was necessary to melt it before it could be dissolved in the gasolene. The priming coat was allowed to dry thoroughly before the asphalt was applied over it, in order that all the gasolene might escape. The use of this priming coat was found to be of the utmost importance. Asphalt placed on it adheres quite tenaciously, while if applied directly to the original surface of the cement concrete, it breaks off readily." (J. C. Trautwine, Jr., Trans. Am. Soc. C. E., Vol. XXXV, page 93.)

Some of the basins connected with the new filtration system of the Philadelphia water-works are lined with asphalt to insure water-tightness of the structures. As a result of experiments made to determine the character of the mixture which would remain upon the slopes without creeping, the following mixtures were used.*

"Floor and first layer of slope:

585 pounds Seyssel mastic,
315 " grit,
50 " refined Trinidad asphalt,
50 " " Bermudez asphalt.

"Second layer of slope:

598 pounds Seyssel mastic,
332 " grit,
33 " refined Trinidad asphalt,
37 " " Bermudez asphalt."

The mixture "containing the larger percentage of bitumen was placed on the floor and first coat of $\frac{3}{8}$ inch on the slopes; a

* Report of the Bureau of Filtration, 1903.

second and finishing coat of $\frac{3}{8}$ inch containing slightly less bitumen was also placed on the slopes." The asphaltic linings were laid $\frac{3}{4}$ inch thick on the concrete linings of the basins. As an additional precaution against movement of the layers on the slopes, the surface of the concrete was grooved.

A small reservoir in Chelsea, Mass., connected with the Metropolitan Works was repaired in 1904 by replacing portions of the defective lining with Portland-cement concrete.* The concrete on the slopes was placed in two layers, the lower being composed of 1 part cement, $2\frac{1}{2}$ parts sand, and $6\frac{1}{2}$ parts crushed stone from $\frac{3}{4}$ inch to $1\frac{1}{2}$ inches in diameter; the upper of 1 part cement, $1\frac{1}{4}$ parts sand, $1\frac{1}{4}$ parts stone-dust, and 4 parts stone. A layer of asphalt about $\frac{1}{4}$ inch thick was placed between the two concrete layers. The asphalt was heated as hot as possible without burning it and poured upon the slopes. Two coats of the hot asphalt were applied. The upper layer of concrete was left about an inch low and finished with a coat of plaster composed of 1 part cement, $1\frac{2}{3}$ parts sand, and $3\frac{1}{3}$ parts stone-dust, which was carefully worked and finished in a manner similar to the method used in making granolithic walks. This plaster coat was applied while the concrete was still moist in order to secure a perfect bond between the layers. The earth bank was well wet before any concrete was placed, and the concrete surfaces were protected with moistened burlap.

To secure the best results with concrete, the sizes of the aggregates and the proportions of the materials used should be such as to insure a dense mass, and the materials should be thoroughly mixed. When dry mixtures are used, thorough ramming is necessary to fill the voids, and owing to this fact concrete is usually placed quite wet in order that the mass may be well compacted. Any plaster or finishing coat should be applied to concrete before the latter has set.

Satisfactory results have been obtained in the waterproofing of walls of gate-chambers and reservoir linings by the application of soap and alum solutions. The method is known as the

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*"Repairs to the Lining of a Small Reservoir on Powder Horn Hill, Chelsea, Mass." C. B. Saville, Jour. N. E. Water-works Assoc'n, Vol. XIX.

Sylvester process. The soap solution is composed of ¾ pound of soap dissolved in one gallon of water, and the alum solution of ½ pound of alum dissolved in four gallons of water. The washes are applied alternately, the soap solution being applied first at a boiling temperature. Twenty-four hours are allowed to elapse between the applications, and these are continued until the desired result is obtained.

In repairing a reservoir of the Pennsylvania Water Company, this process was used in waterproofing the old concrete. Two washes of soap and two of alum were applied at an average cost of about $2.26 per thousand square feet of surface treated.*

* Jour. N. E. Water-works Assoc'n, Vol. XVIII, p. 180.

CHAPTER VII.

MAINTENANCE OF QUALITY.

A water-supply of satisfactory quality and free from pollution by sewage or manufacturing wastes once obtained should be preserved in that condition.

If funds are available for the purpose, the catchment area from which a surface-water supply is derived should be purchased entire by the municipality. Otherwise a strip of land bordering on the storage or impounding reservoirs should be acquired, and such laws as may be enacted by the legislature, or regulations framed by the state board of health regarding pollution, strictly enforced. The location of barns and stables, pig-pens, privies, cesspools, etc., on the borders or in the immediate vicinity of storage-reservoirs and tributary streams should be prohibited. The discharge of polluting material into or deposition near such reservoirs or streams should never be permitted. Bathing in lakes, ponds, or other sources of supply should be prohibited, and boating, fishing, and the gathering of ice should be allowed only under such restrictions as it may be necessary or desirable to impose.

Surface-water supplies are often affected by growths of microscopic organisms which cause disagreeable odors or tastes. Although the presence of such organisms does not, as a rule, impair the healthfulness of the water, the odors or tastes produced by them are at times very objectionable and cause consumers to resort to the use of water from springs or wells for drinking purposes. In many cases the water from these latter sources may be apparently of good quality but in reality, owing to sewage contamination, its use may be productive of serious consequences.

The cause of the presence of these micro-organisms in water-supplies is not at present definitely known, but the extent of their growth appears to depend in large measure upon the amount and character of the organic matter present in the water. This organic matter may be derived from accumulations on the watershed, which are carried into the streams and reservoirs by wind or rain, from swamps on the catchment area, and from the bottoms and shores of muddy ponds or storage-reservoirs which have not been properly prepared for the reception of water.

Water obtained from catchment areas which include little or no areas of swampy land, which is free from contamination by sewage or other objectionable material, and which is retained in storage-reservoirs that have been carefully prepared by the removal of soil and organic matter from the slopes and bottoms, is rarely affected seriously by these growths.

A surface water-supply may be improved in some cases by grubbing and stripping the shores of ponds or storage-reservoirs between the high and low water-levels. Deposits of peat or mud on the bottom of the pond or reservoir should also be covered with a foot or more of clean sand or gravel. Improvement will also result from the filling or draining of stagnant pools and swamps on the catchment area, since these often serve as breeding-places for microscopic organisms. In the drainage of swamps, the water from the uplands is intercepted before it enters the swamp and conveyed in ditches to the reservoir.

A large amount of work of this nature has been carried out by the Metropolitan Water Board, and the methods used are described by Mr. E. S. Larned, C.E., in a paper * read before the New England Water-works Association.

The methods used varied with the character of the swamp; in some instances a single main drain with cross-drains was sufficient, and in others drains entirely surrounding the swamp were constructed. The drains were ordinarily laid in sand or gravel below the peat formation, thus intercepting some ground-water as well as surface-water. A cross-section of a drain as constructed is shown in Fig. 98.

* "The Drainage of Swamps for Watershed Improvement," Jour. N. E. Water-works Assoc'n, Vol. XVI.

"The drain has a board bottom, twelve inches wide, upper side planed, resting on mudsills 2″×4″×2′, spaced about 3 feet apart and set flush with the ground; on either side are wood strips made of 4″×4″ stock, sawed on the diagonal and rabbeted to set over the edge of the board about three-eighths of an inch, serving as a footing for the stone paving on the sides of the channel, and to

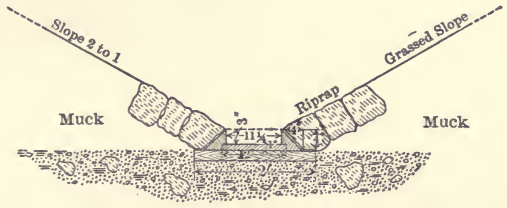

FIG. 98.—Section of Drainage Ditch.
(Jour. N. E. Water-works Assoc'n, Vol. XVI, p. 39.)

hold the boards in place when the nails used may give out through the action of rust."

"The side slopes, two horizontal to one vertical, are protected by paving from 6 to 9 inches thick, up to a point above the normal surface of the water; at all turns and connecting ditches the exposed slopes are protected by paving for the full height; paved gutters are made for influent streams coming in from the side."

The drainage ditches are designed to carry a run-off of $\frac{3}{4}$ of an inch in 24 hours, the maximum velocity of flow being fixed at 2 feet per second.

The depth of the drains or ditches varied from $1\frac{1}{2}$ to 3 feet, with the average depth about 2 feet. The cost of constructing about 9 miles of drains was approximately 28 cents per foot, exclusive of engineering.

The results obtained have been very satisfactory, especially as regards the color of the water. This has apparently been reduced about 30 per cent. Fig. 99 shows the appearance of a swamp before and after the construction of drains.

The character of the water contained in a pond or reservoir of a depth of from about 20 to 200 feet varies greatly during the course of a year, particularly if the bottom is muddy or covered

with deposits of organic matter. During the winter months the water at the bottom approaches the point of maximum density. This occurs at about 39.2 degrees Fahrenheit and the water in the lower layers is warmer than that near the surface, which, although colder, is lighter. When the ice breaks up in the spring the surface layers become warmer until the temperature throughout the mass becomes uniform. More or less circulation of the water takes place at this time. Through the summer months the upper layers of water are warmer than those near the bottom, and the water lying below the level usually affected by wind, twenty to twenty-five feet, remains stagnant. In the autumn the surface-water becomes colder, the temperature of the entire body becomes more uniform, and circulation again takes place. These four periods are respectively termed periods of winter and summer "stagnation" and spring and fall "circulation" or "overturning."

During the periods of circulation the organic and mineral matter and the products of decomposition which have accumulated in the water of the bottom layers as a result of sedimentation, or are dissolved from deposits, are brought near the surface, where they afford a food-supply for microscopic organisms.

If the storage-reservoir is of comparatively limited extent, the water of the lower layers may be drawn off through the waste-pipes previous to the periods of circulation with a view to the limitation of the growth of microscopic life. During the periods of stagnation, the supply may be advantageously taken from the upper layers, if the gate-chamber is provided with intake-valves or ports for that purpose.

The microscopic organisms which cause the most trouble from odors and tastes in surface-water supplies are Asterionella, Anabæna, Clathrocystis, Cœlosphærium, Aphanizomenon, Dinobryon, Peridinium, Synura, Uroglena, and Glenodinium.* The natural odors caused by these organisms and the usual season during which the largest growth of the organisms occurs are shown by the table on page 317.

* Whipple, "Microscopy of Drinking-water," p. 81.

Before Drainage.

After Drainage.

FIG. 99.—Swamp Drainage on the Metropolitan Water-works.

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Organism.	Natural odor.	Season of maximum growth.
Diatomaceæ.		
Asterionella	Aromatic, geranium, fishy..	Spring and autumn
Cyanophyceæ.		
Anabæna	Grassy and mouldy	Summer
Clathrocystis	Grassy	"
Cœlosphærium	"	"
Aphanizomenon	"	"
Protozoa.		
Dinobryon	Fishy	Spring and autumn
Peridinium	"	Summer
Synura	Cucumber, bitter taste	Spring and autumn
Uroglena	Fishy, oily	Winter
Glenodinium	Fishy	Summer

The natural odors are due to oils or other substances contained in the organisms and not to their decomposition. These odors are intensified by the disintegration of the organisms, and many of these are broken up in their passage through the pipes of a distribution system or by heat. On account of this fact the water drawn from the service-pipes, particularly when heated, will often be objectionable, while the odor of the water obtained directly from the source of supply will be only slightly noticeable. The odors due to the decay of these organisms and others not mentioned are much more disagreeable than the natural odors.

The Cyanophyceæ are often termed "blue-green algæ." These organisms form scums upon the surface of the water, and the odors produced when the organism is decomposed are very offensive. At this latter stage in the decay of the organism the odors produced are aptly termed "pig-pen" odors. "Anabæna, Clathrocystis, and Cœlosphærium seldom give trouble unless the temperature of the water is above 70° F." (Whipple, Microscopy of Drinking-water, p. 100.)

When a water-supply is obtained from more than one storage-reservoir it is at times practicable to temporarily shut off the supply-pipes from the one affected by organic growths, and draw the water for consumption from the remaining reservoirs. A similar procedure may be followed when the water in a distribution-reservoir is affected.

Supplies derived from flowing streams without storage rarely give trouble from odors or tastes caused by microscopic organisms.

At the present time little is known with regard to methods for the removal from water-supplies of the odors or tastes produced by micro-organisms. The experiments of the Massachusetts State Board of Health indicate that in some instances single or double filtration through sand may remove the odors and the organisms which produce them.*

A method originating with Dr. Geo. T. Moore appears to be of value if carefully used when the reservoir or other source of supply is not covered with ice.†

This method is known as the "copper sulphate treatment." The copper sulphate is added to the water of the reservoir affected in the proportion of one part of chemical to from one to ten million parts of water; the quantity of chemical used being dependent upon the amount and character of the growth which it is desirable to destroy, and the temperature of the water. The copper sulphate is placed in a coarse bag, which is drawn through the water from a boat rowed back and forth on the reservoir. A knowledge of the specific organism which is the cause of the objectionable odor is necessary in order that this treatment may be intelligently applied. This information may be obtained by a microscopical analysis of the water affected. The best results from the use of this chemical are apparently obtained during the early stages of the growth of the organisms, since, if the growth is well established, a second application of the copper sulphate may be necessary. Furthermore, if an excessive amount of the chemical is used, there is danger that fish, particularly the smaller sizes, will be killed, and the healthfulness of the water may be impaired. This method was used successfully, particularly for the removal of Anabæna, in a number of instances during the summer of 1904.

Trouble from odors caused by microscopic organisms is not usually experienced when the supply of water is obtained from the ground and stored in reservoirs or tanks which are covered so as to exclude the sunlight. Since ground-waters usually

* "Removal of Color, Organisms, and Odor from Water." H. W. Clark, Jour. N. E. Water-works Assoc'n, Vol. XVII.

† Bulletins Nos. 64 and 76, Bureau of Plant Industry, U. S. Department of Agriculture.

contain more or less mineral matter in solution which forms food for plant-life, water of this character is very liable to deteriorate if exposed to the light. Filtered water is also subject to a like deterioration under similar circumstances.

Ground-waters are not, however, entirely free from organic growths, as some organisms do not require sunlight for their development. Of these forms, Crenothrix often causes trouble in water-supplies which are derived from wells driven in swampy land, or which are imperfectly filtered. This organism causes a precipitation of oxide of iron, which makes the water turbid and objectionable for laundry purposes. Conditions favorable to the growth of Crenothrix are often produced by excessive rates of pumping from wells or filter-galleries. Aeration of the water, or treatment with chemicals with subsequent filtration, has been employed for the removal of the iron.

In general the number of organisms found in the water contained in the pipes of a distribution system is smaller than the number present in the water of the source of supply. Growths of Crenothrix or some of the varieties of fresh-water sponge, however, occasionally occur in pipe lines or conduits. Since many organisms require light for their development and existence, and many disintegrate when subjected to pressure or to rapid motion of the water, numbers die and decompose when carried into the pipes of a distribution system. These disintegrated and decomposing organisms tend to collect at low levels and at dead ends, and accumulations of this nature at the latter points, with the accompanying decomposition, result in a diminution or entire exhaustion of the dissolved oxygen in the water. In consequence water drawn from or near dead ends is more liable to be affected by disagreeable odors or tastes, and such odors and tastes persist longer at these points than at others where circulation of the water is possible. On this account dead ends should be avoided whenever possible and those of a system should be thoroughly flushed at frequent intervals. During the summer months, when the temperature of the water is comparatively high, dead ends should be flushed once a week; at other seasons longer intervals of time between flushings may be allowed to elapse.

It is good practice to flush the entire system of distribution once or twice a year, the water being wasted through blow-offs and hydrants.

Sediment or organic matter should not be allowed to accumulate in distribution reservoirs to any large extent. Floating matters will be driven to one side of the reservoir by the wind and may then be removed from the surface by skimming. Reservoirs with slopes paved with asphalt, brick, or concrete are best cleaned by the use of brooms, scrapers, or squeegees. During the operation of cleaning, the valves on the supply- and outlet-pipes are closed, the waste-valve opened, and the water-level gradually lowered in order that the material on the slopes may not become dry before it is removed. The cleaning is performed by men working from a raft which is held against the slope by ropes, and drawn along as the cleaning proceeds. When the bottom is reached, the material is worked toward the outlet and allowed to flow away through the waste-pipe. A hose stream will be found useful at this time to stir up the sediment and keep it in motion, or the material may be scraped into piles and removed with buckets or tubs. The material should not be allowed to become dry, as it then adheres to the surface with which it is in contact, and if the cleaning of the entire reservoir cannot be accomplished in one day, the waste-valve should be closed at the cessation of work and not opened until the work is resumed.

Valuable aid in the maintenance of the quality of a water-supply is afforded by sanitary examinations of the catchment area and the water derived therefrom. A complete sanitary examination of a water-supply consists of five parts: an examination of the catchment area, reservoir, and tributary streams, a physical examination of the water, a microscopical examination, a bacteriological examination, and a chemical analysis. The first is of primary importance, as upon it depends the proper interpretation of the results obtained from the subsequent examinations and analyses. The latter are also dependent in large measure one upon another for their interpretation. After the general characteristics of a water have once been determined, partial examinations and analyses only are usually made, since

the results obtained from certain portions do not warrant the expense involved in their determination.

The funds required for the installation and maintenance of a laboratory equipped with modern appliances for the examination of water are not, as a rule, available, except in the case of large or extensive works supplying districts or the largest cities. However, several States are now following the lead taken by Massachusetts in the establishment, under the control of the State Board of Health, of laboratories and experiment stations of this nature, and the publication of the results obtained therein. The excellent example set by Massachusetts and a limited number of the other States in this respect will probably be followed by other States in the future.

The physical examination of water consists of the determination of its temperature, turbidity, the amount of sediment, color, and odor. The turbidity of a water is due to matter in suspension; the color to coloring-matters extracted from leaves, etc., and held in solution. When an analysis of water is made a sample is allowed to stand about twelve hours and the amount and general character of the sediment noted. The turbidity or color of a water is expressed in arbitrary units, and is determined by comparing the sample with standard solutions. In the former instance similar results are obtained by noting the distance below the surface of the water a platinum wire of standard diameter disappears from view. Highly colored waters are often improved in appearance by storage, with exposure to sunlight, which results in bleaching, or the drainage of swamps on the catchment area. Odors are caused by living organisms, vegetable matter in solution, or organic matter which is undergoing decomposition. The odors vary with the specific organism or matter to which they are due, and are usually intensified by heating the sample.

The microscopical examination is of value for the determination of the cause of the turbidity, color, and odor observed. This examination is of particular value in the latter instance, since the specific organism which is the cause of the odor may be ascertained when the odors observed are due to the presence of microscopic life.

At the present time the bacteriological examination is used principally as a means of gaging the efficiency of purification methods. The bacterial content of a water before and after filtration is determined, and the reduction in numbers, expressed as a percentage, is a valuable aid in the management of filtration-plants. When pollution by sewage or animal wastes is suspected, a water is examined for the presence of the Bacillus coli. This organism is present in the intestines of man and warm-blooded animals, and when found in a water-supply is proof of more or less serious contamination.

The chemical analysis of a water furnishes evidence with regard to the amount and condition of the organic matter, and the quantity of mineral matter contained in the water in question. As nitrogen is an essential element of all organic life, determinations of the nitrogenous matter present are expressed as "albuminoid ammonia," "free ammonia," nitrites, and nitrates. Albuminoid ammonia represents the organic matter present which has not begun to decompose. Free ammonia is usually an indication of the presence of organic matter which is undergoing decomposition. The final stage in the decomposition, or oxidation, of organic matter is represented by the nitrates. The nitrites represent the products of decomposition previous to the final mineralization of the organic matter. Nitrites should not be present in a good ground-water, and only to a limited extent in an unpolluted surface-water. Evidence as to the amount of organic matter present is also given by the determinations of the "oxygen consumed," "total organic nitrogen," and the "loss on ignition." Other determinations are the "hardness" or soap-destroying power of the water, the amount of iron, the "chlorine" content, and the "total residue on evaporation." The "fixed solids," or the remainder of the "total residue" after ignition, is an indication of the amount of mineral matter present in the water.

The chlorine content of a water is of special significance, since salt once introduced into a water always remains therein. A chlorine content in excess of the normal for the locality from which the water is derived affords evidence of either present or past pollution by the waste products of animal life. If lines

are drawn upon a map through the points where the normal chlorine, that is the chlorine content of an unpolluted water, is the same, a series of lines termed "isochlors" are obtained. Isochlors have been established as a result of systematic examinations and analyses in several States.

An excessive amount of chlorine, free ammonia, and nitrites is usually an indication that the supply is contaminated by sewage or other objectionable wastes, and that such contamination is recent. An excessive amount of chlorine and nitrates, in general, indicates that the water has been polluted and subsequently purified.

The amount of foreign material present in a water as determined by a chemical analysis is usually expressed in parts per million or per one hundred thousand.

CHAPTER VIII.

WATER WASTE.

At the present time the problem of reducing the waste of water, which invariably occurs in all unmetered cities and towns, is receiving an amount of attention from engineers and water-works officials equal to that formerly given to the problem of securing additional supplies, when the demands for water bid fair to exceed the capacity of the source in use.

The modern sanitary requirement that water supplied for domestic consumption shall be free from disease-producing organisms, or pollution which may give rise to the presence of these organisms, results in a more rigid investigation of proposed sources of supply, and consequently many are now considered unsafe which formerly would have been accepted without question. In such instances resort must be made to methods of purification, and the disastrous experience of many communities with polluted supplies has also made purification desirable, if not an urgent necessity, in many existing works.

The records of consumption in various cities and towns indicate that the amount of water wasted often far exceeds the quantity necessary to satisfy the demands of health or convenience. This waste results largely from the failure of the majority of consumers to appreciate the value of a supply of pure water. The installation of plumbing fixtures of inferior grade, which remain tight but a short time, the failure to keep pipes and fixtures in good repair, and carelessness in the use of water impose an unnecessary burden on the works. A continuation of these conditions entails increased expenditures for additional supplies, new mains, pumping machinery, purification works, etc., with a corresponding increase in the annual expenditures for their main-

tenance and operation, and often necessitates the postponement of measures designed for the improvement of supplies in use.

A large proportion of the waste of water is preventable, especially that portion which has its origin in the erroneous belief of many consumers that water is free, and, therefore, to be used without regard to economy. This misuse of the supply is due to the custom of many consumers to allow water to run freely: in the winter to prevent the pipes from freezing, in the summer that the water may be cool for drinking, at all times under the impression that the drainage system is kept cleaner, and to the neglect of leaky pipes and fixtures in order that a plumber's bill may be saved. The loss of water through inferior fixtures, the leakage of which passes unnoticed or is disregarded by the average consumer, and the excessive use on lawns in suburban districts, still further increase the volume of waste on the premises of the water-taker.

In this respect the municipality is often equally at fault in permitting the excessive use of water in public buildings, especially schools, in drinking and ornamental fountains, for street-sprinkling and sewer-flushing.

Still further losses are due to leakage from reservoirs, from the numerous joints of the pipes composing the supply, distribution, and service systems, and from small breaks, the leakage from which does not appear on the surface. In general these losses are practically non-preventable.

The loss occasioned by the slippage of pumps, referred to in a previous chapter, is, strictly speaking, not a loss of water, but of power.

The amount of water required for domestic purposes, in which may be included that used on lawns and in private stables, varies from five to fifty gallons per person per day, and is dependent chiefly upon the character and mode of life of the individual consumers. The water used for manufacturing purposes is usually, or should be, metered, and therefore need not necessarily be considered in all investigations of waste. The following statistics regarding the consumption of water in various cities and towns are of interest and value in the consideration of this subject.

METERED CONSUMPTION FOR THE YEAR 1902 IN CITIES AND TOWNS WHERE METERS ARE LARGELY USED.*

City or town.	Estimated number of consumers.	Per cent of services metered.	Total daily average metered consumption. Gallons.	Gallons per consumer per day.		
				Domestic.	Manufacturing.	Totals.
Brockton, Mass.	37,800	90.0	822,670	13.2	6.5	21.8[1]
Fall River, Mass.	107,650	96.0	2,607,100	15.5	5.2	22.2[1]
Newton, Mass.	35,400	86.0	1,202,740	23.1	4.6	34.0[1]
Ware, Mass.	7,690	100.0	203,430	25.4	1.1	26.5
Woonsocket, R. I.	34,474	86.7	762,380	11.6	10.5	22.1
Wellesley, Mass.	5,147	100.0	132,540	25.6	0.1	25.7
Worcester, Mass.	119,330	94.5	4,331,030	16.1	17.8	36.3[1]
Yonkers, N. Y.	51,000	98.7	2,413,380	19.7	22.0	42.7[1]
	398,491		12,475,270	16.7		

[1] Includes water used for public purposes.

CONSUMPTION OF WATER THROUGH METERS IN BELMONT, MALDEN, MILTON, AND WATERTOWN, MASS.*

City or town.	Number of consumers.		Gallons per day.		Gallons per day per consumer.	
	1901.	1902.	1901.	1902.	1901.	1902.
Belmont	3,600	3,900	63,760	66,630	17.7	17.1
Malden	21,100	22,550	414,030	450,160	19.6	20.0
Milton	6,850	7,450	115,000	143,500	16.8	19.3
Watertown	9,650	10,250	147,200	152,900	15.3	14.8
	41,200	44,150	739,990	813,190	18.0	18.4

"In the towns of Belmont and Milton every service-pipe is metered, in Watertown 89.5 per cent, and in Malden 63.4 per cent. The above quantities include water used for stables supplied in connection with dwellings and that used for lawn-sprinkling, as well as that used strictly for household purposes. In all these municipalities the meters have been in use but a comparatively few years, and it is not probable that they have become worn so as to cause a large percentage of error in registration." *

*"Report on the Measurement, Consumption, and Waste of Water Supplied to the Metropolitan Water District." Dexter Brackett, C.E., Jour. N. E. Water-works Assoc'n, Vol. XVIII.

USE OF WATER IN TENEMENT-HOUSES IN BOSTON DURING 1902.*

Ward.	Number of houses.	Number of families.	Number of persons.	Gallons per day.		Monthly rental.
				Per family.	Per capita.	
Ward 6.....	21	263	1226	115	24.8	$12 to $16
Ward 7.....	8	75	365	71	14.5	12 to 16
Ward 7.....	13	155	755	64	13.2	16 to 20
Ward 8.....	15	131	625	113	23.7	16 to 20
Ward 9.....	20	171	758	92	20.7	12 to 20
Ward 8.....	12	116	553	141	29.6	20 to 25
Ward 8.....	10	88	420	138	29.0	25 to 30
Ward 10.....	21	209	949	150	33.1	25 to 30
Ward 10.....	20	250	1135	200	43.9	30 to 40
Ward 11.....	10	111	545	190	38.7	25 to 45
Ward 11.....	10	89	437	194	39.5	35 to 55
Ward 11.....	9	81	398	218	44.4	50 upwards
	169	1739	8166	139	29.63	

DOMESTIC CONSUMPTION PER CAPITA IN NEWTON, FALL RIVER, WORCESTER, AND LONDON, ENG., AS DETERMINED BY METER MEASUREMENT.*

City or town.	Number of houses.	Number of families.	Number of persons.	Consumption. Gallons.		Remarks.
				Per family.	Per capita.	
Newton.....	490	490	2450	132.5	26.5	All houses supplied with modern plumbing
Newton.....	—	619	3005	—	6.6	These families have but one faucet each
Newton.....	—	278	1390	34.5	6.9	These families have but one faucet each
Fall River...	28	34	170	127.5	25.5	The most expensive houses in the city
Fall River...	64	148	740	42.0	8.4	Average class of houses, generally having bath and water-closet
Worcester...	—	81	327	80.2	19.9	Woodland Street, best class of houses
Worcester...	—	37	187	118.1	23.4	Cedar Street, best class of houses
Worcester...	—	93	447	95.0	19.8	Elm Street, houses of moderate cost
Worcester...	—	245	1104	55.1	12.2	Southbridge Street, cheaper houses
Worcester...	—	229	809	55.0	15.6	Austin Street, cheaper houses
London, Eng.	1169	—	8183	—	25.5	Houses renting from $250 to $600; each have bath and two water-closets
London, Eng.	727	—	5089	—	18.6	Middle class, average rental, $200

*"Report on the Measurement, Consumption, and Waste of Water Supplied to the Metropolitan Water District." Dexter Brackett, C.E., Jour. N. E. Water-works Assoc'n, Vol. XVIII.

In localities where water is paid for at meter rates and a minimum rate established, the records of the consumption in cases where a less quantity of water is used than the consumer is entitled to under the minimum charge furnish data with regard to the actual requirements for domestic purposes. In 1888 an analysis of the accounts of consumers in the city of Providence who paid only the minimum rate—this minimum being $10.00 per year, for which amount the use of 91.32 gallons per day was permitted—showed the following results: *

	Per annum.	Gals. per day per tap.	Which, at 5 persons per family,
167 families drew less than	1500 cu. ft.	= 30.742	= 6.15 gals. per capita
237 families drew the previous amount but less than	2000 cu. ft.	= 40.989	= 8.20 gals. per capita
361 families drew the previous amount but less than	2500 cu. ft.	= 51.236	= 10.25 gals. per capita
445 families drew the previous amount but less than	3000 cu. ft.	= 61.484	= 12.30 gals. per capita
446 families drew the previous amount but less than	3500 cu. ft.	= 71.731	= 14.35 gals. per capita
462 families drew the previous amount but less than	4000 cu. ft.	= 81.978	= 16.40 gals. per capita
435 families drew the previous amount but less than	4457 cu. ft.	= 91.324	= 18.27 gals. per capita

A similar analysis of accounts of the year 1901 in the city of Malden, Mass.—where the minimum rate is $12.00, for which amount the use of 130 gallons per day is permitted—showed the following results: †

Cubic feet per year.	Number of services.	Gallons per day per service.	Cubic feet per year.	Number of services.	Gallons per day per service.
Less than 500	28	[1] 6.4	Between 3000 and 4000	566	72.0
Between 500 and 1000	52	[1]16.1	Between 4000 and 5000	560	92.0
Between 1000 and 2000	259	[1]33.9	Between 5000 and 6300	621	114.0
Between 2000 and 3000	441	52.5		2527	77.4

[1] Many of these were metered for but a portion of the year.

Mr. Dexter Brackett has engaged in many investigations of the consumption and waste of water in Boston, Mass., and vicinity,

* J. H. Shedd, "Requisite Amount of Water for a Public Supply," Jour. N. E. Water-works Assoc'n, Vol. XVIII, p. 8.

† Jour. N E. Water-works Assoc'n, Vol. XVIII, p. 146.

and his estimate of the quantities required for all legitimate purposes by a community is as follows:

Domestic use	25.0	gallons per capita per day
Manufacturing, mechanical, and trade use	23.5	" " " " "
Public use	7.0	" " " " "
Total	55.5	" " " " "

The estimate of the quantity required for public use is divided as follows:

Public buildings	3.78	gallons per capita per day
Drinking and ornamental fountains	1.00	" " " " "
Street-sprinkling	2.13	" " " " "
Flushing water-pipes and sewers and extinguishing fires	0.20	" " " " "
Total	7.11*	" " " " "

The statistics which have been given refer to the average consumption; the actual daily consumption is subject to more or less fluctuation throughout the year, the departures from the average being usually more marked in unmetered communities. The average consumption may be exceeded 50 per cent or more during days in summer when water is used extensively on lawns, and in winter when water is wasted to prevent freezing.

In all metered cities and towns the amount of water registered by the service-meters is less than the measured amount delivered into the mains, the discrepancy amounting to from 20 to 50 per cent of the quantity supplied. This discrepancy is probably due to errors in the various measurements caused by the slip of pumps, by the under-registration of meters, and to losses through leakage from the mains and services and, in some instances, from unauthorized connections. According to various authorities the losses due to leakage from the distribution system should not exceed about 3000 gallons per day per mile of main. However, this figure is often exceeded, especially where comparatively large leaks occur without any surface indications. In 1902 four such leaks were discovered in the course of a waste investigation at Stoneham, Mass., which wasted about 470,000 gallons per day; similar investigations in Boston have disclosed

leaks amounting in single instances to from 24,000 to 100,000 gallons per day.

The following instances afford illustrations of the extent to which water may be wasted by a municipality. In the spring of 1903 it was found that the consumption of water in the public schools of Syracuse, N. Y., was at the rate of about 20 gallons per pupil daily, the range being from 2 to 497 gallons.* In contrast to these figures the consumption of water in metered school buildings in the Boston metropolitan district was found to vary from 2.2 to 7 gallons per pupil per school day. Sewer flush-tanks are in many cases of doubtful value, and in any case should not use more than 300 to 500 gallons per day, but 57 such tanks in Richmond, Ind., were found using 223,370 gallons daily, the discharge from one tank being at the rate of 17,280 gallons in that time.†

Where meters are not extensively used the larger portion of the waste of water takes place on the premises of the consumers as a result of negligence or wilfulness on the part of the latter. "The greatest source of negligent waste from defective fixtures is undoubtedly the ball-cock which controls the flow of water into tanks supplying water-closets and other fixtures. The ball-cock seldom remains tight more than a few months, and when defective allows a constant stream of water, often of considerable size, to flow unseen, though not always unheard, to the sewer."‡ The losses due to a leaky ball-cock may often be limited only by the capacity of the service-pipe, and the drippings from faucets may amount to 100 gallons per day and upwards in single instances.

Investigations of Waste.

In all investigations of waste it is essential that means be available for the determination of the amount of water supplied

* "Municipal Use and Waste of Water." John Venner, Jour. N. E. Water-works Assoc'n, Vol. XVII, p. 270.

† "Results of an Investigation of the Amount of Free Water Supplied to the City of Richmond, Ind." H. A. Dill, Proc. Am. Water-works Assoc'n, 1903.

‡ Brackett, "Report on the Measurement, Consumption, and Waste of Water Supplied to the Metropolitan Water District," Jour. N. E. Water-works Assoc'n, Vol. XVIII, p. 134.

and the variation in the flow during twenty-four hours. In the case of a direct-pumping system computations may be based upon observations of the indications of the pump-counters, but the results obtained are, however, often considerably in error, owing to the slip of the pumps. Where stand-pipes or distributing reservoirs of a capacity equal to the daily consumption are available, data may be obtained by shutting off all other sources of supply and noting at intervals the decrease in height of the water surface of the stand-pipe or reservoir, from which the total consumption and the consumption during each hour of the day may be determined. In the latter instance, however, the figures relating to the total consumption may be affected by leakage from the reservoir, but the differences in the hourly rates should not be appreciably affected thereby. Where the

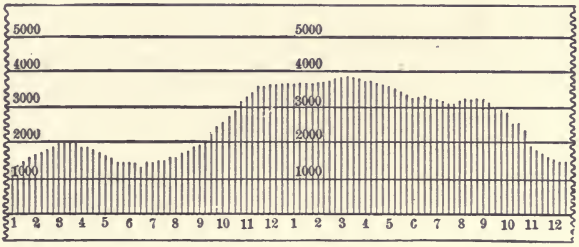

FIG. 100.—Venturi Meter Chart.

reservoir is of comparatively large extent and the drop of the water-surface small, observations of the latter are of little value.

The total consumption may be ascertained by a Venturi meter provided with a register, and the rates of flow at intervals during the twenty-four hours by this apparatus with the addition of a chart-recorder, or by a Venturi tube and manometer. The Venturi meter has been described in Chapter I; the manometer consists of a U tube partly filled with mercury, one side of the apparatus being connected by a pipe with the inlet pressure-chamber of the meter-tube and the other by a second pipe with the throat pressure-chamber. When water flows through the meter-tube the inequality of the pressures at the inlet and the throat causes a difference in the height of the mercury columns in the U tube, which is measured by a sliding scale graduated to indicate the rate of flow corresponding to these differences.

The results obtained with this instrument are limited by the number of observations made.

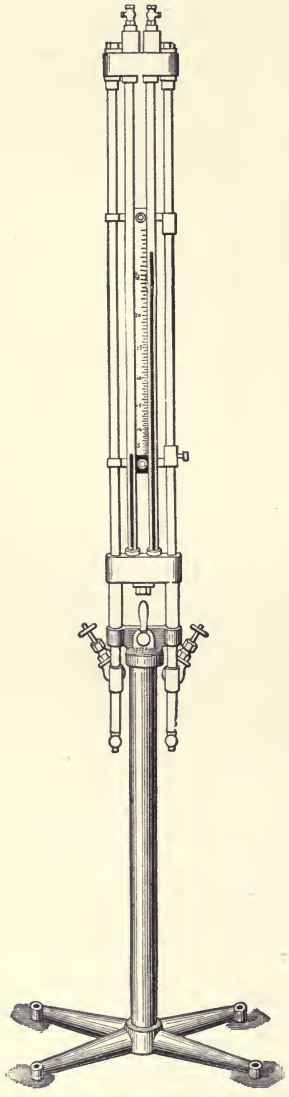

FIG. 101.—Manometer.

The Deacon waste-water meter has been used in the course of waste investigations in some English cities and in Boston for the determination of the rates of flow. This meter is set on a by-pass, which with the main pipe-line is provided with valves by which the flow may be diverted through the meter during experiments.

"The cast-iron body of the meter contains a hollow, cone-shaped chamber of brass. Suspended in this chamber *C* is a composition disc *D* of the same diameter as the upper end of the cone. The fine wire by which the disc is suspended passes through a small hole in the top of the chamber over the pulley *R*, and is attached to a counterweight *W*. When the water in the pipe is at rest the disc is held at the top of the cone by the counterweight, but when water is drawn through the meter the disc is pressed downward, its position depending upon the quantity of water passing through the meter. By means of a pencil attached to the wire the motion of the disc is recorded upon a drum, *E*, revolved by clockwork once in twenty-four hours. The meter does not, like the ordinary house meter, record the total quantity which has passed between any two observations, but it records upon a diagram the rate of flow at each and every instant."*

* Dexter Brackett, C.E., "Consumption and Waste of Water," Trans. Am. Soc. C. E., Vol. XXXIV, p. 198.

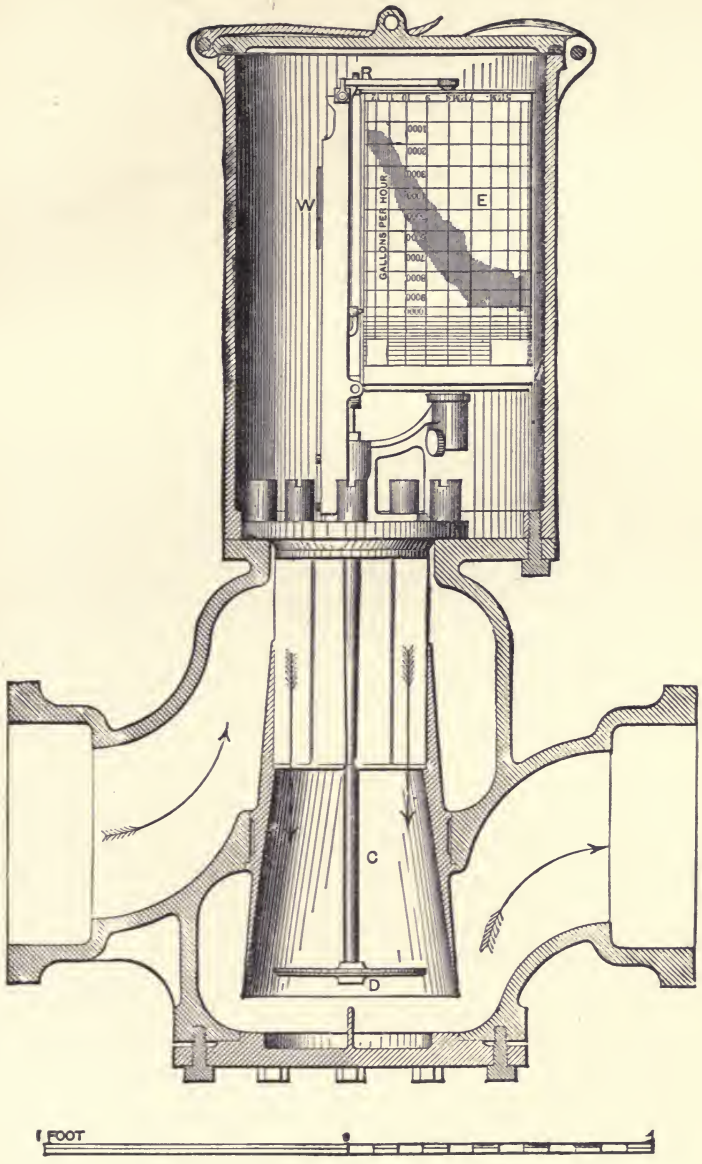

FIG. 102.—Deacon Waste-water Meter.
(From Trans. Am. Soc. C. E., Vol. XXXIV.)

The meters used in Boston were 6 inches in size, the registering capacity being about 25,000 gallons per hour. This type of meter on account of its size is not well adapted for use in measuring the flow in large mains.

Measurements of the flow of water in a pipe with a Venturi or Deacon meter require the cutting of the main pipe, and when in place the meter cannot be conveniently removed during the course of an investigation. These objections may be overcome by the use of a Cole-Flad pitometer, an instrument which has

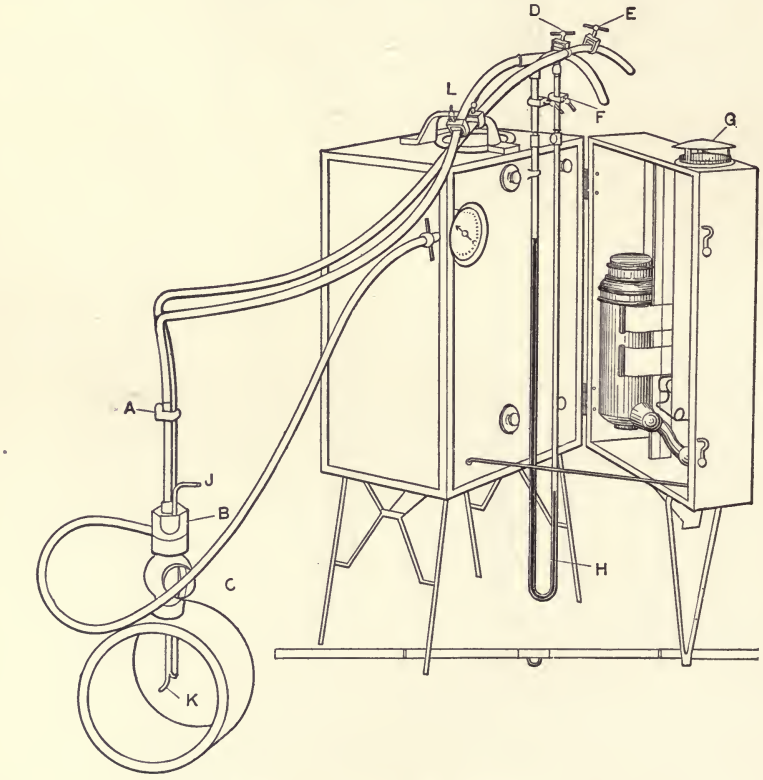

FIG. 103.—Cole-Flad Pitometer.

been largely used in recent investigations of waste in New York and other cities.

"This instrument is primarily a rate-meter, depending as it does upon the velocity of the water within the main. . . . Two

brass tubes bent at their lower ends with carefully formed orifices of $\frac{1}{8}$ inch internal diameter are held in a suitable cap which screws upon a standard 1-inch corporation cock through which the tubes may be readily introduced into any main and as easily withdrawn. Heavy cloth-insertion rubber tubing connects the orifice tubes with a long glass manometer or U tube, and blow-off cocks are provided to remove air from the instrument. The U tube is half filled with a mixture of carbon tetrachloride and gasoline having a specific gravity of 1.25, and when in use the water from the pipe fills all the space in the U tube and connections. The orifices are set to receive the maximum velocity within the main, which is usually near the center, and is readily

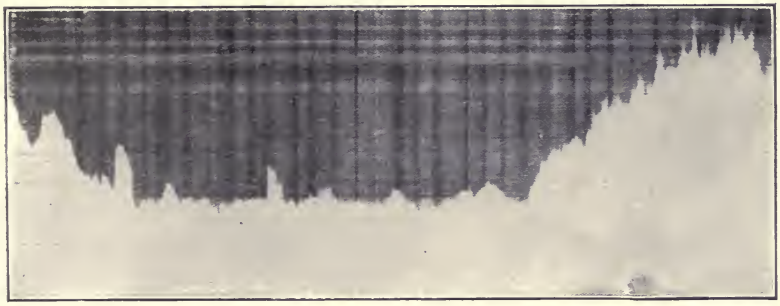

FIG. 104.—Pitometer Record.

indicated by the deflection in the manometer. This deflection, by virtue of the differential action of the water and the slightly heavier and insoluble liquid, is just four times that due to the actual difference of water-head on the orifices produced by the flowing stream. The current impinges directly on one orifice, but the other is turned down-stream and gives something less than the static head within the main, thus magnifying the difference of pressure produced. This difference is then multiplied in the U tube, the result being that a low velocity within the pipe produces a readable deflection. Without this effect the Pitot tube would hardly indicate a velocity less than one foot per second, while the pitometer may be depended upon, if carefully used, to measure velocities as low as 4 inches per second. At the ordinary velocities the U-tube deflections may run up to 24 inches and need

not be read with any great nicety. The photo-recorder consists of a box in which a drum-carrying Velox paper revolves before a fine vertical slit, just in front of which is locked one leg of the U tube in such a position that the rays of light from an oil-lamp will be partly intercepted on their way through the colored liquid in the lower half of the manometer. As the liquid rises and falls with the velocity in the pipe it will record a line or band of shade on the Velox paper whose ordinates vary according to the well-known formula

$$V = c\sqrt{2gh},$$

in which h is one-quarter of the U-tube deflection in feet or half the recorded ordinate on the paper. A 12-hour or 48-hour photographic record is 18 inches long and 10 inches high. Autographic horizontal lines are formed by notches in the drum-slit. spaced so as to correct for the angularity of the light and enable the deflection to be readily taken from the diagram at any points

"The accuracy of the pitometer has been established by many weir tests and by calibration in open channels where the absolute velocities indicated by the instrument were compared with float measurements.

"The calibration constant c of the orifices has thus been experimentally determined to be .800 in our formulas.

"We then have

$$V = .8\sqrt{\frac{2gd}{4}} = 3.212\sqrt{d},$$

in which d is the U-tube deflection in feet. The actual velocity of the water is indicated in this way wherever the orifices may be placed within a pipe, and by completely traversing a given pipe section on one or two diameters we may easily compute the ratio of mean to maximum velocity at some one rate of flow. This ratio once determined for a given pipe is the same at all velocities. One may then set the orifices permanently to give the maximum or center velocity within the pipe, reading off the mean velocity and the discharge from a table constructed for the particular size of pipe and velocity ratio."*

* Edward S. Cole, C.E., "Water Waste and its Detection." Read Oct. 22, 1902, before the Western Society of Engineers, Chicago.

Mr. Cole considers that "the flow may be determined by the pitometer method within 2 or 3 per cent of the truth in fairly clean pipes when the velocity exceeds 1 or 2 feet per second."

The first step in an investigation of waste is the determination of the probable amount of water wasted or lost, considering the community as a unit. In few instances is water required for legitimate purposes between midnight and four o'clock in the morning, and the rate of consumption, as determined by the use of one of the methods of measurement heretofore mentioned, during the intervening hours is usually a good indication of the amount of water hourly wasted and lost through leakage. Having ascertained these figures the next step is the determination of the distribution of the waste. For this purpose the city or town is divided into convenient sections, and by closing the valves in the mains, section after section is cut off, the time when a section is shut off and the indication of the measuring device being noted. In like manner the waste may be further localized by closing valves in the mains supplying the various streets. The distribution of the waste on the streets may be best determined by closing all the valves on the mains leading to the street in question and supplying the latter through one or more lines of hose laid from a hydrant on an adjoining street to a hydrant on the main which is shut off. The amount of water supplied may be measured by a service-meter placed in the hose line. The losses from the service connections beyond the curb, and the leakage from the street mains and services to the curb cocks, may be differentiated by closing the latter and noting both time of closing each and the reading of the meter.

The loss due to leakage from the latter portion of the system cannot, however, be determined with accuracy if the valves in the mains are not tight. Leakage through a valve will be manifested by a hissing sound, which is audible to a person holding an ear to the shut-off rod, when the latter is in contact with the valve-stem. This hissing sound will be magnified by the use of a waterphone, an instrument somewhat resembling an ordinary telephone-receiver.

The losses in some streets may be so small that it will not be necessary to carry the work through to the extent indicated

the records obtained by the first three methods outlined indicating which streets may or may not be examined in detail.

The actual work involved in an investigation of waste is usually not as simple as the foregoing brief outline may, perhaps, indicate, particularly that in connection with the determination of the sub-divisions of the waste. Old and leaky valves, abandoned service connections which have not been shut off at the curb or corporation cock, unauthorized or surreptitious connections, the

FIG. 105.—Aquaphone.

absence of curb cocks on many service connections in use, etc., introduce complications and increase the cost of and time required for the experiments. The value of a complete set of plans and records will be appreciated at this time if not before.

Waste Prevention.

The methods used for the prevention of waste have been limited in practice to the following:

1. The extensive use of service-meters.
2. House-to-house inspection.
3. The Deacon system.

The first method requires no explanation, the second merely locates and attempts to check the waste beyond the curb cock on the premises of the consumers, and the third indicates the waste substantially by the methods heretofore outlined and in connection with the second attempts to check it.

In the house-to-house inspection system of waste prevention, inspectors test the service connections at night for leakage by listening, with the aid of a waterphone and gate-wrench, at the

curb cock for the hissing sound which is an indication that water is passing through the latter. The sound is intensified by closing the cock slowly, and is made more evident in doubtful instances by closing the cock entirely and then partly reopening it. If no leakage is manifested, no further examination of the premises is made, but if it is evident that water is being used or wasted the fact is reported, and an inspector examines the interior piping and fixtures during the day. If leaks or defective fixtures are found the property-owner is notified and requested to place the plumbing in good repair.

The inspection methods have been found expensive, a source of annoyance to the occupants of suspected houses, a cause of more or less friction between the property-owner and the water department, and unless persisted in the results obtained are not of permanent value.

On the contrary, the use of meters has proved satisfactory to both consumers and department in all works where these appliances have been installed. The use of meters results in the co-operation of both parties in the prevention of waste when the water is furnished at meter rates instead of schedule rates. "It does this immediately, yet without need of argument or urging. The mere establishment of an instrument to keep a record of amounts drawn, and the presentation of quarterly bills based on such amounts, is all that is needed. The bills convey their own lesson, and it is not possible to convey such a lesson without such bills." *

The value of the water-meter as a waste preventive agent is very forcibly illustrated by the diagram (Fig. 106), prepared by Mr. Freeman, for his report on "New York's Water-supply."

Further illustrations are afforded by the following statistics, which, in connection with those previously given, serve as the best arguments in favor of the use of water-meters for the prevention of waste.

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* Clemens Herschel, C.E., "An Attempt to Prove that Thrift in the Operation of Water-works is more Economical than Waste." Proceedings Am. Water-works Assoc'n, 1901.

EFFECT OF THE USE OF METERS UPON THE CONSUMPTION OF WATER IN VARIOUS CITIES.

Compiled by J. R. FREEMAN, C.E

Quantity shown in gallons per inhabitant per day. Showing also the effect of meters in restraining the natural increase of consumption in recent years. The curve *A B C* is not an exact average, but is drawn as summing up the general tendency and as a convenient guide to the eye in studying the comparative effect of various percentages of meters in the different cities. Obviously the first 5 or 10 per cent of meters, if judiciously applied to the worst cases of suspected waste, effect a much larger saving than any further equal number of meters.

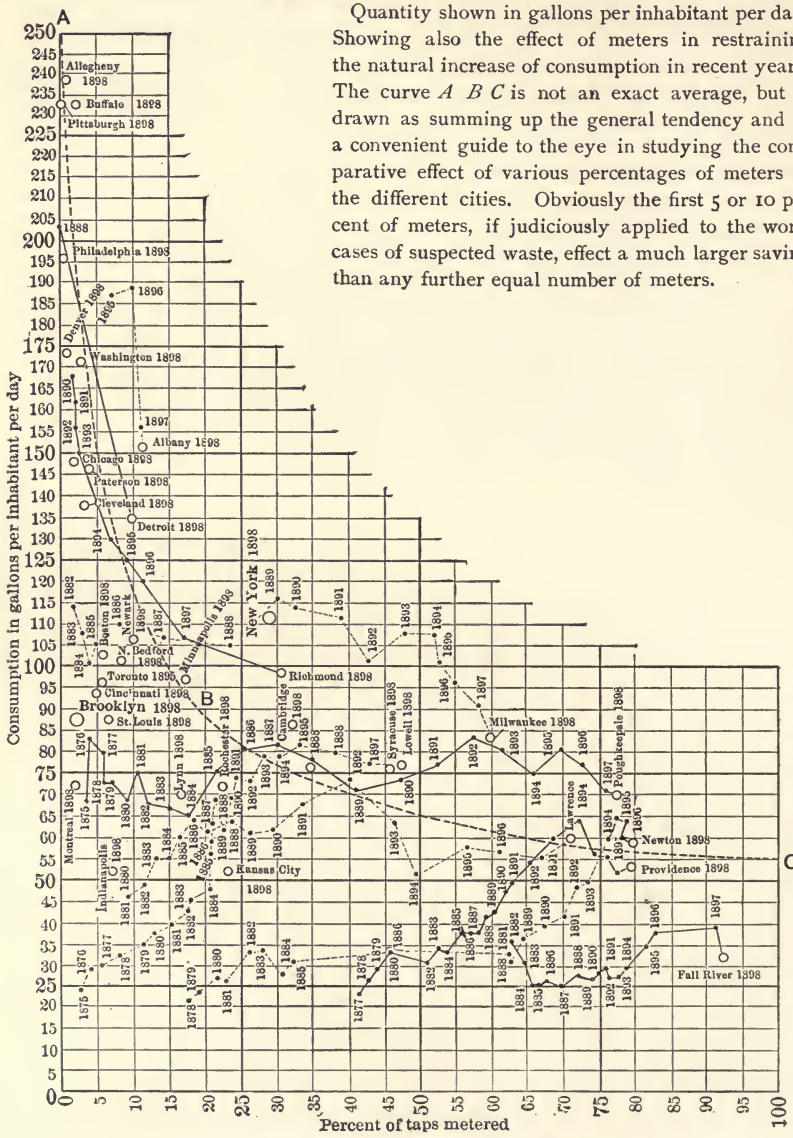

FIG. 106.---Effect of the Use of Meters

Nevertheless service-meters, useful as they are, do not reduce losses due to leakage from the street mains and services, and do not entirely eliminate waste on the premises of the consumer. The former losses may be reduced by careful work in the construction of extensions and service connections, and by the repair of breaks and defects in the system. Losses due to the under-registration of service-meters may be reduced by more rigid requirements as regards the quality of the plumbing fixtures to be connected with the works.

POPULATION, WATER CONSUMPTION, AND RECEIPTS FOR WATER SUMMARIZED AND AVERAGED IN ACCORDANCE WITH PERCENTAGE OF TAPS METERED.*

Percentage of taps metered.	No. of cities.	Population.†	Consumption, gallons per day.		No. of cities.	Population.†	Receipts.	
			Total.	Average per consumer.			Total.	Average per consumer.
0 to 10	71	12,713,866	1,947,378,000	153	53	11,802,154	$26,432,316	$2.24
10 to 25	18	1,657,454	182,991,000	110	12	1,108,142	2,491,719	2.25
25 to 50	22	1,686,635	175,632 000	104	17	1,448,633	3,255,149	2.25
50 to 100	23	1,600,333	109,616.000	62	20	1,455.442	3,253,606	2.24
Totals and averages	134	17,658,288	2,415,617,000	137	102	15,814,371	$35,432,790	$2.24

* Geo. I. Bailey, C.E., "The Effect of Water-meters on Water Consumption in the Large Cities of the United States," Engineering News, April 18, 1901.

† The table is based on the population supplied where separately reported: otherwise on the total population by the census of 1900. The cities included have populations of more than 25,000.

AVERAGE DAILY CONSUMPTION OF WATER PER PERSON IN 1900 IN CITIES AND TOWNS OF MASSACHUSETTS, ARRANGED IN GROUPS ACCORDING TO THE PERCENTAGE OF SERVICES WHICH ARE PROVIDED WITH METERS.*

	Gallons per person.
Average of 14 cities and towns in which more than 75 per cent of the services are metered	39.8
Average of 24 cities and towns in which more than 25 per cent but less than 75 per cent of the services are metered	47.1
Average of 31 cities and towns in which not more than 25 per cent of the services are metered	59.7

* "Consumption of Water in Cities and Towns in Massachusetts," Report of State Board of Health, 1900, p. 608.

A Comparison of the Per Capita Consumption of Water in Cities and Towns where Meters are in General Use (A) with that in those where Water is Paid for at Schedule Rates (B).*

(Compiled by Dexter Brackett, C.E., 1903.)

A.

City or town.	Number of consumers.	Per cent of services metered.	Consumption (gals. per day per consumer).
Milwaukee, Wis	308,000	80.0	81.0
Providence, R. I.	198,400	84.5	58.0
Worcester, Mass.	119,330	94.5	68.0
Fall River, "	107,650	96.0	41.0
Lowell, "	100,000	65.0	57.0
Lawrence, "	65,000	83.0	53.0
Brockton, "	37,800	90.0	36.0
Newton, "	35,400	86.0	54.0
Woonsocket, R. I.	34,474	96.0	29.0
Ware, Mass.	7,690	100.0	44.0
Wellesley, Mass.	5,147	100.0	49.0
Reading, "	4,385	100.0	33.0
Totals	1,023,276	....	61.7

B.

City or town.	Number of consumers.	Per cent of services metered.	Consumption (gals. per day per consumer).
Buffalo, N. Y.	360,000	2.0	324.0
Indianapolis, Ind.	169,100	6.0	79.0
New Haven, Ct.	108,000	2.6	150.0
New Bedford, Mass.	61,000	18.0	104.0
Cambridge, "	94,150	15.0	85.0
Haverhill, "	37,200	10.0	95.0
Lynn, "	74,000	25.0	63.0
Waltham, "	24,550	6.0	99.0
Salem, "	36,250	3.0	79.0
Montague, "	6,150	2.0	73.0
Dedham, "	7,500	2.0	83.0
Braintree, "	5,980	1.0	91.0
Totals	983,880	....	178.5

In the above table "an attempt has been made to eliminate . . . as far as possible" differences due to the effect upon the consumption caused by "the differences in climate, in character of business, and in location, as affecting the available supply of water."

* Jour. N. E. Water-works Assoc'n, Vol. XVIII, p. 143.

CHAPTER IX.

ELECTROLYSIS.

THE recent wide-spread introduction of street-railways using electricity as a motive power presents a new problem in the maintenance of water-works, that of electrolysis. In the usual construction of railways of this kind the single-trolley system is used; the current is conveyed from the positive side of the generator by the overhead wires to the trolley on the car, thence to the operating motor, and then returns to the power-house through the rails, return-feeders, water-pipes, or other underground metallic structures, and the earth to the negative side of the generator. This return-current is divided in accordance with the established law that the amount of current which is carried by a number of conductors is distributed inversely as the resistance of the several paths.

In many cases the resistance offered by the soil between the rails and the water-pipes and the piping system is comparatively small and considerable current is diverted to the water-mains, where serious damage occurs at the points where the current leaves the pipes. At the points where electrolysis occurs, the conditions are similar to those which obtain in an electrolytic cell. The pipe corresponds to the positive plate of the cell, the rail, or other metallic structure, to the negative plate, and the moisture in the soil holding in solution salts of the alkaline earths which are readily decomposed by a current of electricity forms the electrolyte. A difference of potential of about one and one-half volts is required for the electrolysis of water, but the salts ordinarily found in solution in the moisture present in soil are decomposed by a potential difference of a small fraction of a volt. In this latter instance the metal of the pipe is attacked by the

acid radicals produced by the decomposition of the soluble salts. To these secondary chemical reactions a large proportion of the disintegrating effect of the current is due.

As a result of electrolytic action the metal of the pipes is pitted or softened. The softening often occurs to such extent that the pipe can be cut with a knife or nails driven into the pipe wall. The pipes are reduced in thickness and strength until ultimately they burst and more or less damage and inconvenience results. The effect of electrolysis upon a cast-iron water-pipe is shown in the accompanying illustration from the annual report of the City Engineer of Providence, R. I., for the year 1903.

FIG. 107.—Four-inch Cast-iron Water-pipe, showing Effect of Electrolysis.

The amount of damage caused by stray electrical currents depends upon several conditions. One is the nature of the salts present in the soil. Professor Jackson concludes as a result of experiment and investigation that the order of activity of these salts is as follows: chlorides, nitrates, sulphates.* Another condition is the character of the material of which the pipes are composed. "With a given current density and potential cast iron is affected the least, wrought iron the next, mild steel the next, high carbon steel the next, and lead the most." † The extent of the damage is also dependent upon the current strength (amperes) and the time during which the flow takes place. The current also depends upon the electrical pressure or potential difference, and the resistance of the soil between the pipes and

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*"The Corrosion of Iron Pipes by the Action of Electric Railway Currents," Jour. Assoc of Eng. Soc., Vol. XIII.

† Report of Albert B. Herrick, E.E , to the City Engineer of Rochester, N. Y., 1903.

the rails. The drier the soil the more resistance it offers to the passage of the current.

The amount of leakage from the rails and other return conductors provided by the street-railway companies depends upon the character of the construction of the road and the amount of traffic. Owing to this fact the amount of current diverted to the water-pipe system may be no larger in a city where improved methods of construction are used than in a suburban locality where the traffic is comparatively small but the conductivity of the track return very poor.

Service-pipes, particularly those of lead, which pass under or near street-railway tracks usually present the first indications of electrolytic action. It was observed in Peoria, Ill., that about 96 per cent of the breaks in service-pipes occurring on streets where car-tracks were laid took place in instances where the pipes passed under the tracks.* In this instance it was also observed that the metal removed from the lead service-pipes was redeposited in other form on the particles of gravel and earth lying in the path of the current from pipe to rail.

The points at which the most serious injury to the pipes takes place, or the so-called "acute" points, are where the current leaves the pipes. These are usually in the vicinity of power-stations, switches, and turn-outs, or near points where the street-railway tracks cross those of a steam road. As the injury to main pipes is in general first evidenced at such points, it was formerly deemed advisable to connect the water-mains with the rails or negative side of the generator. Although this method of procedure proved beneficial so far as local damage to the pipes at the points where the connections were made was concerned, it was found that the method resulted in an increased flow of current through the pipe system.

Later investigations showed that the cast-iron pipes of a system were not continuous conductors, and that the pitting and wasting away of the pipe previously observed at the acute points also occurred at the joints in the pipe-lines. Although the lead

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* D. H. Maury, "Electrolysis of Underground Metal Structures," Proc. Am. Water-works Assoc'n, 1900.

used in making the joints between lengths of cast-iron pipe is a conductor of electricity, the protective coating applied to the pipes serves as an insulating medium, and the heat given off by the molten lead when a joint is poured is not sufficient to destroy this coating. Experiments made by Mr. D. H. Maury, C.E., and others indicated that the resistance offered by the joints of the pipe lines tested amounted to upwards of 90 per cent of the total resistance of the line. In consequence of this joint resistance a large part of the current carried by a cast-iron main is shunted around the joints and passes from one pipe to the next through the water within the pipe and the moist soil without. The pipes are, therefore, pitted at the points where the current leaves the different lengths, as shown by the accompany-

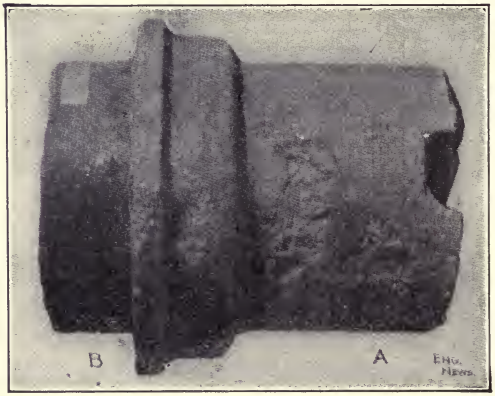

FIG. 108.—Pittings at Joint in 12-inch Cast-iron Water-main. Current Flowing from *A* to *B*. (From Eng. News, Vol. XLIV, p. 41.)

ing illustration from the paper by Mr. Maury, to which reference has previously been made. Wasting of the lead in the joints also occurs, and in consequence a joint so affected in time leaks and eventually fails by the blowing out of the lead.

Connections between water-pipes and rails or other return conductors are now considered very inadvisable except as a temporary expedient.

That the damage occasioned by the passage of currents of electricity through and from the pipes of a water-works system is often of considerable moment is evidenced by the numerous

cases of injury to, or destruction of, pipes and appurtenances reported during the past ten years. Aside from the question of the cost of replacing and repairing pipes and appurtenances so injured, considerable damage may be caused to adjacent property by the sudden breakage of large mains. Consumers may also be seriously inconvenienced if supply or large distribution mains are affected, or the value of a system for fire protection may be at times greatly reduced.

Since the cause of electrolytic damage is not under the control of the water department, measures for the prevention or diminution of such injury should be undertaken by, and at the expense of, the street-railway companies. However, investigations and tests should be undertaken by the water department, in localities where electric roads are in use, in order that data with regard to the existence or probability of such damage may be approximately ascertained, and if conditions favorable to electrolysis obtain, the matter should be brought to the attention of the parties responsible therefor.

As an aid in the investigation of this subject a map or plan should be prepared upon which the location of water-pipes and appurtenances, street-railway tracks, and power-stations is shown. Measurements are then made of the difference of potential between pipes and rails at convenient points, and also to ascertain the approximate current carried by the pipes. The measurement of potential difference is made with a portable voltmeter, with one lead in contact with the rail and the other with the water-pipe or an adjacent hydrant. An instrument with the zero in the center of the scale is to be preferred, since positive or negative readings are recorded without the necessity of changing connections; or two voltmeters may be coupled together, the positive readings taken from one and the negative readings from the other. For convenience, voltmeters provided with two or three scales should be used. A millivoltmeter will be required to measure very small differences of potential.

The voltmeter measurements indicate the direction of the current flow and the electrical pressure or difference of potential. When a positive reading is indicated by a voltmeter with the

positive binding-post in electrical connection with a pipe or hydrant, and the negative with the rail or other metal in the ground, current will be found to be flowing from the pipe. On the contrary, if the pipe is found to be negative to the rail the current is flowing to the former.

The readings of a voltmeter will fluctuate with the movement of the trolley-cars and the observations made at a given point should be extended to cover the various conditions depending upon the positions of the cars. The size of the voltmeter readings at a given point will also vary with the varying amount of moisture at different periods of the year. Any metallic connection between pipes and rails will tend to reduce the size of voltmeter readings.

The results obtained from the observations with the voltmeters should be noted upon the map or plan, and notes with regard to the conditions obtaining at the time the readings were taken should be preserved. The areas in which the pipes are negative to the rails, those in which the pipes are positive, and those where they are alternately positive and negative may then be approximately determined.

At or near the points where the highest positive readings are obtained (pipe to rail), excavations should be made and the pipes examined for evidence of electrolytic action. Accumulations of rust and scale should be removed from the exterior of the pipe, the depth and size of the pits, and the degree of softness of the metal noted.

Measurements of the current flow should also be made where the pipe is exposed in the trench. The direct method of measurement is seldom applicable, as this would necessitate the cutting of the pipe, and joining the ends with a copper wire, with an ampere-meter in the circuit. Usually the measurement of the current flow is made by the "drop-of-potential" method. This method is based on Ohm's law, that the current varies directly as the electromotive force and inversely as the resistance of the circuit or $C=\frac{E}{R}$, when $C=$amperes of current, $E=$volts, and $R=$ohms of resistance. The measurement should preferably be made on a straight piece of pipe between joints, and the con-

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tacts of the instrument leads with the pipe should be carefully made. The drop of potential is measured with a millivoltmeter. The resistance of the pipe is an unknown factor, but its approximate value may be determined by experiments upon similar pipes. In this instance currents of known strength are caused to flow through the pipe under test and the corresponding drop of potential in a given distance measured. Tables may then be prepared from which the current flow in different sizes of pipe corresponding to the observed drop of potential may be ascertained; or special tests for the determination of the resistance of cast-iron pipe may be dispensed with and the current flow corresponding to the observed drop of potential taken from the tables prepared by Mr. D. H. Maury, C.E. The table for use with cast-iron pipe is as follows:

CURRENT FLOW IN CAST-IRON PIPE WITH ONE MILLIVOLT DROP.*
(From tests showing average resistance per pound per foot = 0.00144 ohm.)

Inside diameter of pipe in inches.	Approximate weight per foot without bell.	Distance in feet between points of contact on pipe.									
		1	2	3	4	5	6	7	8	9	10
		Current on pipe per millivolt drop.									
4	20	13.9	6.9	4.6	3.5	2.8	2.3	2.0	1.7	1.5	1.4
6	30	20.8	10.4	6.9	5.2	4.2	3.5	3.0	2.6	2.3	2.1
8	39	27.1	13.5	9.0	6.8	5.4	4.5	3.9	3.4	3.0	2.7
10	58	40.3	20.1	13.4	10.1	8.1	6.7	5.8	5.0	4.5	4.0
12	84	58.3	29.1	19.4	14.6	12.0	9.7	8.3	7.3	6.5	5.8
16	120	83.3	42.0	28.0	21.0	17.0	14.0	12.0	10.0	9.3	8.3
20	180	125.0	63.0	42.0	31.0	25.0	21.0	18.0	16.0	14.0	12.5
24	220	153.0	76.0	51.0	38.0	31.0	25.5	22.0	19.0	17.0	15.0
30	310	216.0	108.0	72.0	54.0	43.0	36.0	31.0	27.0	24.0	22.0
36	440	306.0	153.0	102.0	76.0	61.0	51.0	44.0	38.0	34.0	32.0
42	560	389.0	195.0	130.0	97.0	78.0	65.0	56.0	49.0	43.0	39.0
48	720	500.0	250.0	167.0	125.0	100.0	83.0	71.0	62.0	56.0	50.0
60	900	624.0	312.0	208.0	156.0	125.0	104.0	89.0	78.0	69.0	62.0

*"Surveys for Electrolysis and their Results." D. H. Maury, Proc. Am. Water-works Assoc'n, 1903.

When the above table is used, care should be taken that pipe bells or joints are not included between the points of contact of the leads from the instrument.

The following method for the approximate determination of the flow of current is given in Herrick's Electric Railway Handbook. Wires from a voltmeter and an ampere-meter are inde-

pendently connected with points on the water-main. Readings of the voltmeter are taken with the circuit through the ampere-meter open, and also with this circuit closed. At the time the latter voltmeter readings are taken the ampere-meter is also read.

Then $X:A::V_1:V_1-V_2$ or $X=\frac{AV_1}{V_1-V_2}$,

where X = normal current flow on pipe,

A = current diverted through the ampere-meter circuit,

V_1 = voltmeter reading with ampere-meter circuit open,

V_2 = " " " " " closed.

If the connections of the hydrants to the mains are electrically good, approximate results may be obtained by this method by connecting the leads from the instruments with adjacent hydrants on the same main. Owing to the uncertainty with regard to these connections, however, the experiments should be made on several sets of hydrants, or upon a comparatively short length of pipe exposed in an excavation.

In all tests the surface of the pipe should be brightened with a file and care taken to secure good electrical connections between the instrument leads and the pipe. The primary investigations are preferably made under the direction of an experienced engineer, but subsequent periodical examinations may be made by employés of the department provided with suitable instruments for that purpose.

Conditions favorable to electrolytic action as regards water-pipes are not found when the return-current of a street-railway system is completely removed from the ground. The practicable method by which this removal may be effected is the adoption by the street-railway companies of the double-trolley system. This method is not regarded with favor by the railway corporations, since new problems of construction and maintenance and considerable financial outlay are involved in a change from existing conditions. Although the injury to water-pipes and appurtenances resulting from the operation of electric railways by the single-trolley system is now an established fact, such legal actions as have been instituted for the purpose of compelling electric-railway corporations to change from the single- to the double-trolley system have not as yet accomplished the end sought.

The flow of the current from the rails to the water-pipes where single-trolley systems are in use may be reduced by the adoption of better methods of track construction, such as drainage of the sub-grade, the use of heavier rails, and improved methods of bonding, and the installation of overhead return-circuits connected to the rails at frequent intervals. Rail bonds often work loose and as a result the efficiency of the track return is greatly impaired. The electrical condition of the pipe system should, therefore, be examined at intervals by the methods mentioned.

It is often assumed that serious damage will not occur if the readings obtained in voltmeter tests do not exceed one volt, but even in these instances considerable injury may result in the course of time, particularly if the soil surrounding the pipes is moist and impregnated with mineral salts, and the pipes are composed of metal which contains many impurities.

Care should be taken in the location of new mains to place the pipes, valve-boxes, and hydrants as far as possible from the rails. On the other hand, when tracks are laid in streets which contain water-pipes, the rails should be placed at a distance from the pipes. The location of tracks directly over and parallel to water-mains should be avoided if possible. Cast-iron water-pipes may also be advantageously laid with the bells toward the direction in which the current will flow, as the wasting effect due to electrolytic action will then be confined to the heavier and larger bell ends instead of the spigot ends of the pipes.

Several methods have been suggested by various writers for the protection of service-pipes which pass beneath railway tracks, such as enclosing the service-pipe in asphalt, placing the pipe in a rubber hose, or wrapping it with tape or other insulating materials. When main or service-valve boxes are in close proximity to the rails, a portion of the box may be replaced with vitrified pipe. Where the flow of current is from branching mains to a large supply or distribution main, insulating joints have been made in the branch connection as a means of partially protecting the larger and more important pipes. These joints consist of two special castings which are separated by a heavy rubber gasket.

CHAPTER X.

FIRE PROTECTION.

The capacity of a system of water-works, particularly that of the smaller distribution mains, is usually determined in large measure by the requirements for adequate protection against fire. When works are first constructed, the amount of water supplied for domestic and manufacturing purposes is usually but a comparatively small proportion of the total capacity of the system; but as the business of the department increases, this amount likewise increases, and the quantity of water under suitable pressure for the purpose of extinguishing fire becomes less. The character of the service may, also, change as the commercial districts expand, so that although the provision for fire protection was satisfactory at the outset, the increased demands for domestic and manufacturing purposes, together with the additional requirements for fire protection in the commercial and the thickly settled districts, result in rates of draft which reduce the available pressures below the points at which satisfactory service can be given in emergencies.

A diminution of the efficiency of the works in case of fire should be guarded against by measures taken to restore or maintain this efficiency, provided the financial condition of the works admits of the necessary expenditures for that purpose. Much may be done by the installation of additional hydrants, the reinforcement of the system by the laying of circuit lines, and the renewal of some of the older and smaller distribution mains with pipe of larger diameter. When the pressure is inadequate after measures have been taken to improve the hydrant service, dependence is usually placed upon steam fire-engines when large conflagrations occur.

The efficiency of an existing system of works as a means of protection against fire may be measured approximately by certain general rules or standards. This efficiency is also at times indicated, often to the surprise of persons who have based their ideas with regard to the capacity of the works upon static hydrant pressures, when large demands are made upon the works in the case of a serious fire.

The proper allowance for a good fire stream is placed by different authorities at from 175 to 250 gallons per minute. That suggested by Mr. John R. Freeman, C.E., is a stream of 250 gallons per minute, delivered through a $1\frac{1}{8}$-inch smooth nozzle, with a pressure at the base of the nozzle of about 45 pounds per square inch.* With this pressure a 1-inch smooth nozzle will deliver about 200 gallons per minute, a $\frac{7}{8}$-inch smooth nozzle about 150 gallons, and a $\frac{3}{4}$-inch smooth nozzle about 110 gallons. A stream of from 150 to 200 gallons per minute will usually prove satisfactory where fires occur in residential districts, the smaller quantity probably being satisfactory where buildings are so situated that a fire in one does not greatly endanger adjacent property.

The number of good fire streams which should be furnished simultaneously from a system is dependent upon the conditions which obtain in a given instance more than upon the population or valuation. The liability of considerable loss from fire is greater and the need of protection more if in the compact parts of the city or town the buildings are largely of wood. The width of streets, the character of the industries, the nature of the equipment of the fire department, whether or not steam fire-engines are available, and the presence or absence of streams from which such engines may derive their supply are factors to be taken into account in the consideration of this question. Information upon this subject given by previous experience in the handling of large fires in the municipality is also of value.

The number of fire streams of 250 gallons per minute each, based upon the population of the protected community, which

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* John R. Freeman, "The Arrangement of Hydrants and Water-pipes for the Protection of a City against Fire," Jour. N. E. Water-works Assoc'n, Vol. VII.

should be available simultaneously is considered by Mr. Freeman to be as follows:*

Population.	Number of streams.
1,000	2 to 3
5,000	4 " 8
10,000	6 " 12
20,000	8 " 15
40,000	12 " 18
60,000	15 " 22
100,000	20 " 30

The following recommendations are made by the same authority: "So far as a general statement may apply, . . . the pipes should be large enough and the hydrants numerous enough so that two-thirds of the above number of streams could be concentrated upon any one square in the compact, valuable part of the city or upon any one extremely large building of special hazard. Better than by data based on population, the question of the number of streams which it should be possible to concentrate at any one point, as well as the question of the total number of fire streams to be provided for the city, can be best solved by a tour around the given city, studying out the spots where a large number of streams would be needed to check a conflagration which may be conceived to have so got beyond control as to hold some one of the largest buildings in flames from top to bottom and from end to end."

"Ten streams or as large a proportion thereof as the financial consideration will permit may be recommended for a compact group of large, valuable buildings irrespective of a small population."

Fig. 109 † shows the number of fire streams, based on population, considered necessary by different authorities.

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* See previous reference.

† Prepared from table and diagram, page 15, of "The Financial Management of Water-works." E. Kuichling, Trans. Am. Soc. C. E., Vol. XXXVIII.

It should be borne in mind that the quantities of water to be allowed for purposes of fire protection are in addition to the consumption for domestic, public, and manufacturing purposes.

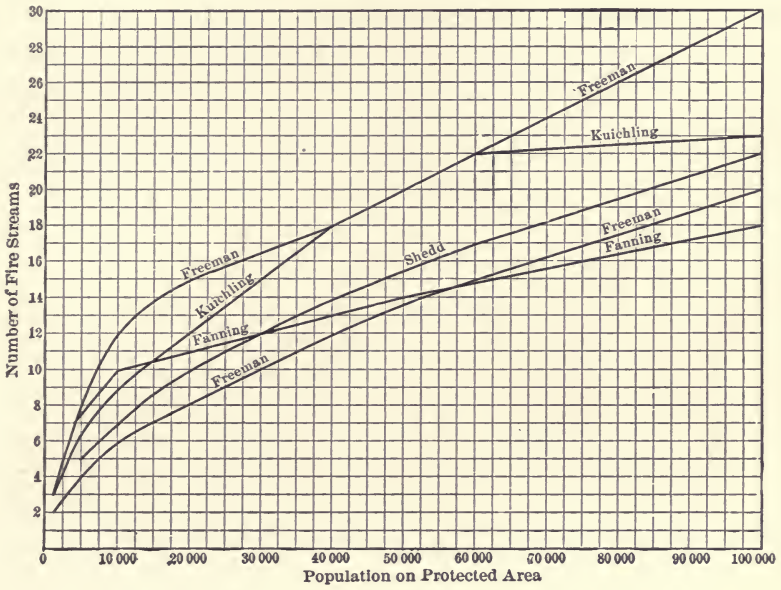

FIG. 109.—Number of Fire Streams Required in Cities.

In general, the amount of water required for fire protection is added to the maximum hourly or daily rate of consumption for other purposes, and provision is made for a total draft equal to this amount for a period of from one to six hours.

The fire-hose in general use is $2\frac{1}{2}$ inches internal diameter, and the comparatively high velocities of flow when water is discharged through a nozzle at rates of from 175 to 250 gallons per minute result in a large loss in pressure owing to the energy consumed in overcoming the frictional resistance of the hose. A like loss of energy occurs when water is drawn in large quantities from long lines of small pipe.

In general, hydrants should be so placed that hose lines of a length greater than 500 feet are not required in residential districts, and of not more than about one-half that length in business or manufacturing districts. The hydrants in a residential

district, where in the majority of instances two good fire streams are sufficient, are usually placed about 500 feet apart; but since it is advisable to place these appurtenances at street intersections, no fixed rule with regard to the distance between hydrants can be followed. When more than two streams are required the average lengths of hose used ordinarily exceed the figures stated. If 45 pounds be taken as the pressure at the base of the nozzle, the pressures required at the hydrants when the streams are flowing, and the best quality smooth rubber-lined hose of $2\frac{1}{2}$ inches internal diameter is used, are about as follows:*

Length of hose.	Size of smooth nozzle.			
	¾-inch.	⅞-inch.	1-inch.	1⅛-inch.
	Pressure required at hydrant.			
50 feet.....	47 lbs.	49 lbs.	52 lbs.	56 lbs.
100 "	48 "	51 "	56 "	63 "
200 "	51 "	57 "	65 "	77 "
300 "	54 "	62 "	74 "	92 "
400 "	57 "	67 "	83 "	106 "
500 "	59 "	72 "	91 "	120 "

Since the pressures at the hydrants usually range from 75 to 100 pounds in the majority of instances, it follows that good fire streams cannot under ordinary conditions be delivered through more than 300 to 400 feet of $2\frac{1}{2}$-inch hose. The hydrants in the commercial district should, therefore, be arranged with this fact in view, and the opportunity to concentrate the number of streams deemed necessary for efficient protection should be afforded.

In cases where long lines of hose are necessary, or the hydrant pressures low, much of the energy used in forcing water through the small hose lines may be saved and made available at the nozzles by the use of two lines of hose, which are connected with a Siamese coupling to one or more lengths to which the nozzle

* John R. Freeman, "Some New Experiments and Practical Tables relating to Fire Streams," Jour. N. E. Water-works Assoc'n, Vol. IV; also "Experiments relating to Hydraulics of Fire Streams," Trans. Am. Soc. C. E., Vol. XXI.

is attached. This saving is due to the fact that by the use of two lines from a hydrant the velocity of flow is decreased and likewise the friction losses in the hose.

Long lines of 4-inch pipe cannot, under ordinary conditions, be depended on to furnish a supply for more than one good hose stream, and in some cases not even one strong stream can be made available. With a given loss of head due to friction, the discharging capacity of a 6-inch cast-iron pipe is about three times, and that of an 8-inch pipe six times, that of a 4-inch pipe of equal length and under similar conditions. The loss of head in new, clean cast-iron pipe, due to friction, is about as follows for pipe of the diameters mentioned:

Discharge, gallons per minute.	Loss of head in feet per 1000 feet.		
	4-inch.	6-inch.	8-inch.
100	7.5	1.0	0.27
200	27.0	3.8	0.93
300	58.0	8.0	1.96
400	97.0	13.5	3.35
500	.....	20.5	5.10
600	.....	28.5	7.00

The advantage of using 6-inch pipe for street mains instead of 4-inch is clearly indicated by the above table, and pipe of larger diameter than 4 inches should be used for mains whenever possible. The minimum sizes of pipe usually adopted for residential districts are 6- and 8-inch, and for commercial districts, 10- and 12-inch.

The diagrams (Figs. 112 and 113) will be found useful in a study of the efficiency of a system as a means of protection against fire. The diagrams relating to the discharge of smooth nozzles and the friction losses in hose are based upon the experiments of Mr. Freeman, to which reference has been made. The pressures at the nozzles and at the hydrants are the pressures at these points when streams are flowing. If the elevations of the nozzle and the hydrant are not the same, an allowance for the difference in height should be made.

The diagrams showing the friction losses in cast-iron pipes are based on computations made by the Hazen-Williams formula:*

$$v = cr^{0.63}s^{0.54}0.001^{-0.04},$$

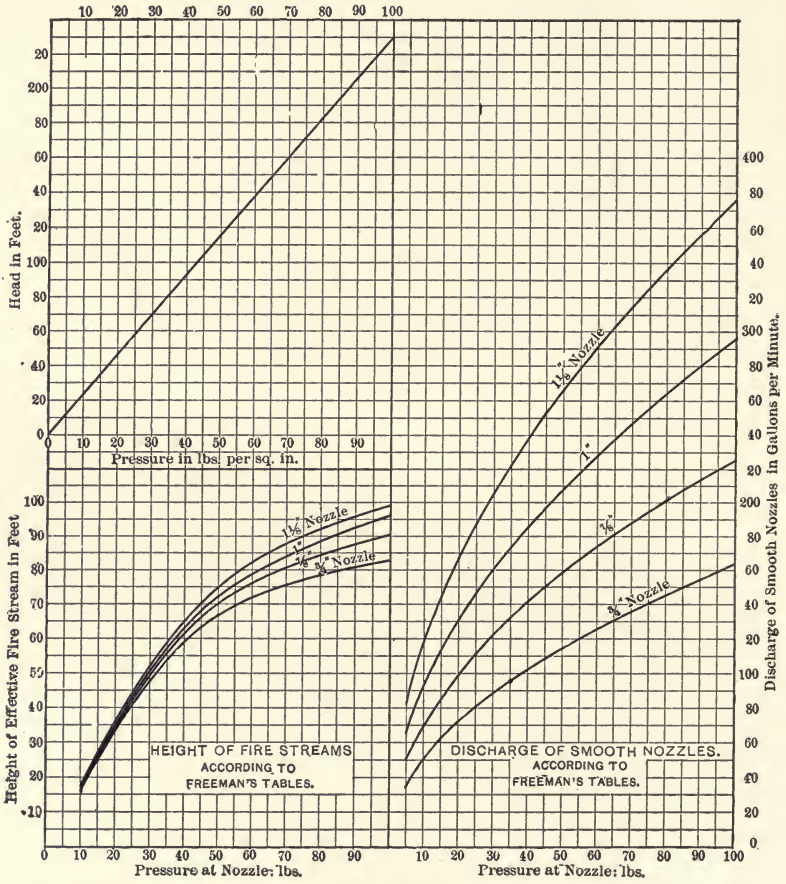

FIG. 110.—Height of Fire Streams and Discharge of Smooth Nozzles.

in which v = velocity of flow in feet per second;

c = coefficient of roughness, which has been taken as 130

* More complete diagrams based on this formula and comparisons with other formulæ and data are given in a paper "Hydraulic Diagrams and Further Notes Upon the Hazen-Williams Slide Rule," by Leonard Metcalf, C.E., Eng. Record, Vol. 47.

for new, clean pipes and 100 for old and tuberculated pipes;

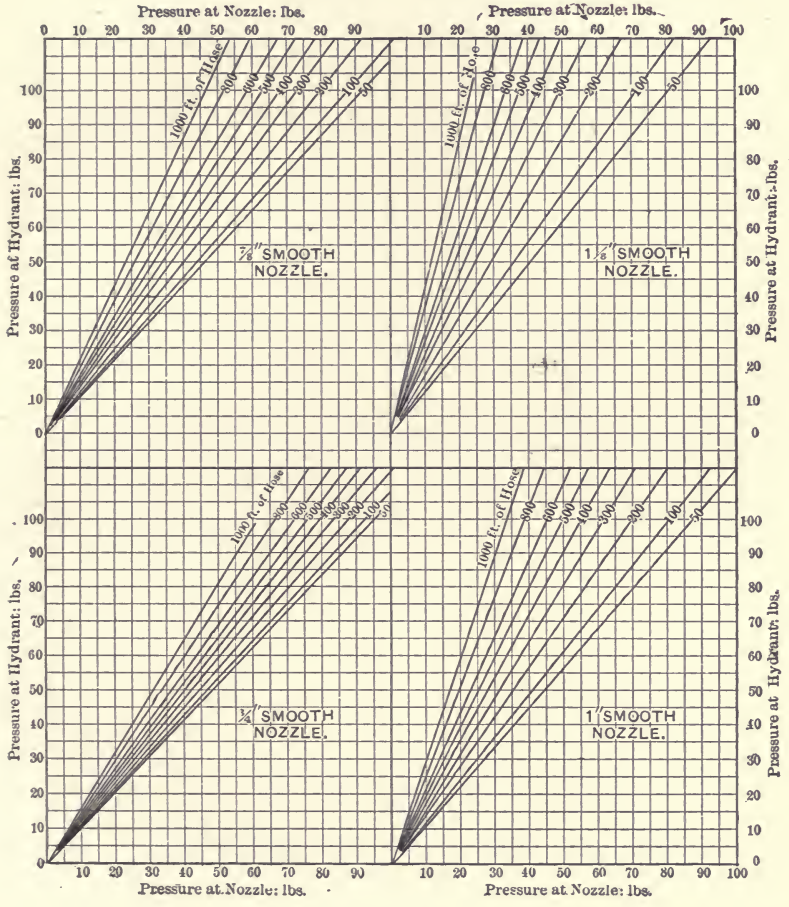

FIG. 111.—Friction Losses in Ordinary Best Quality Smooth Rubber-lined 2½-inch Hose.

r = hydraulic mean radius = $\frac{\text{diameter in feet}}{4}$ for circular pipes;

s = slope = $\frac{\text{head}}{\text{length}}$.

The following example illustrates the use of the diagrams. Assume that a stream of 150 gallons per minute is required from a $\frac{7}{8}$-inch smooth nozzle at the end of 300 feet of hose. Then the pressure at the nozzle should be 45 pounds, and if the nozzle is 20 feet above the hydrant, the pressure at the hydrant should be 62+9, or 71 pounds. If the hydrant is supplied by a 6-inch pipe 2000 feet long, the pressure in the main to which the 6-inch line is connected should be about 73 pounds.

Where hydrants are located on mains supplied through several pipes of different diameters, laid at different times, computations of friction losses consume considerable time, and the results obtained are often very uncertain. A better plan in such cases is to experiment on the ground by attaching a pressure-gage to the main or a hydrant and noting the indications at varying rates of draft.

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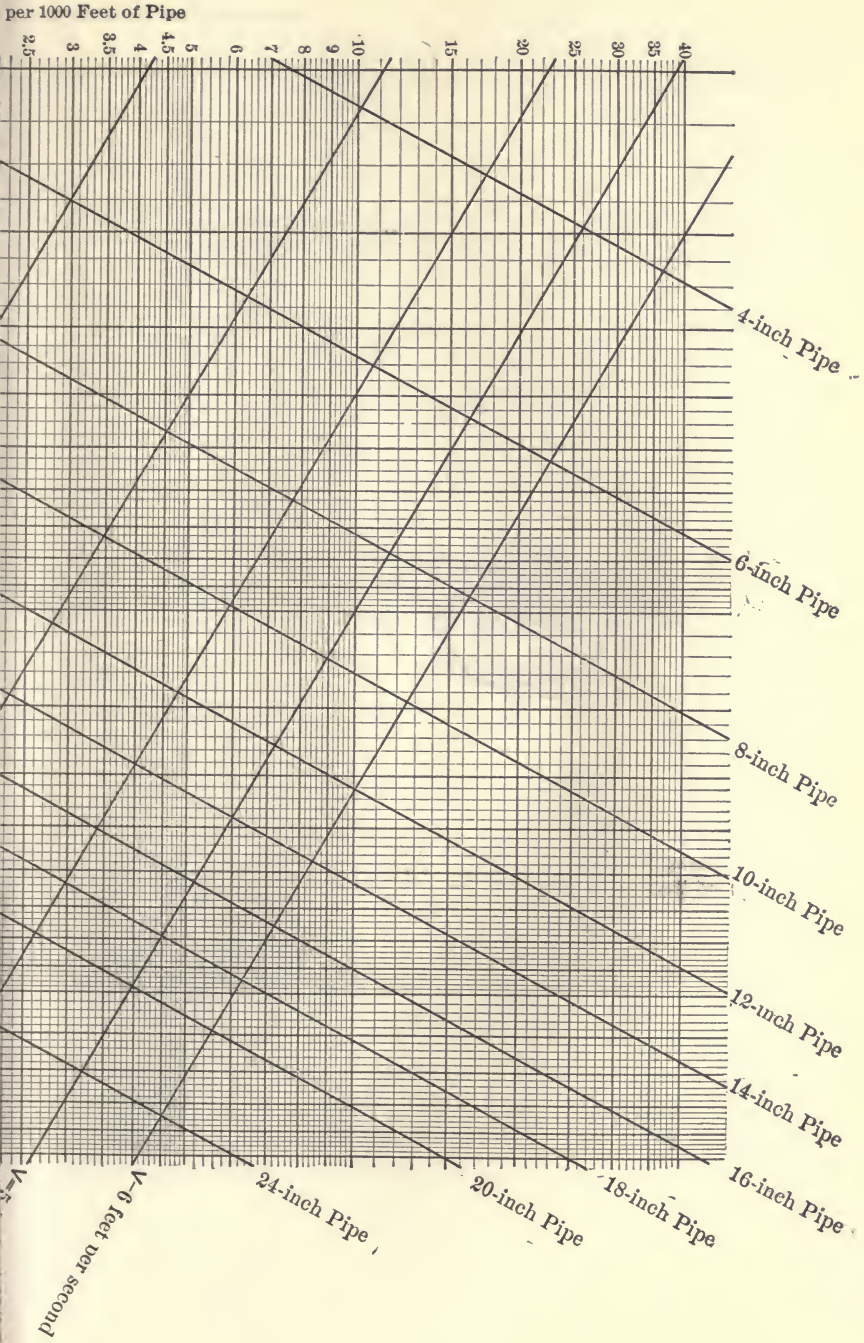
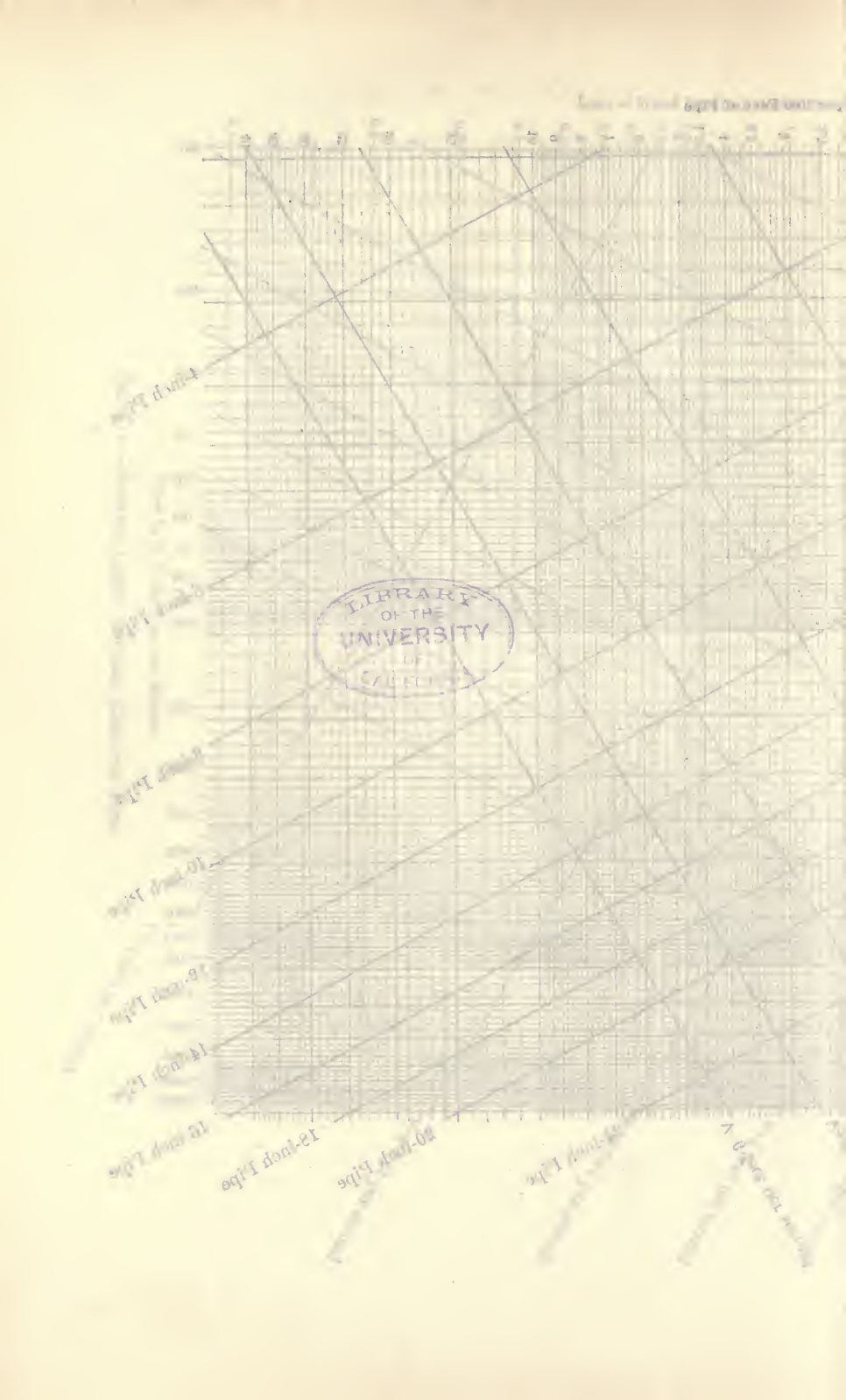
FIG. 112.—Friction Losses in Clean Cast-iron Pipe.


per 1000 Feet of Pipe
2.5
3
3.5
4
4.5
5
6
7
8
9
10
15
20
25
30
35
40
4-inch Pipe
6-inch Pipe
8-inch Pipe
10-inch Pipe
12-inch Pipe
14-inch Pipe
16-inch Pipe
18-inch Pipe
20-inch Pipe
24-inch Pipe
V–6 feet per second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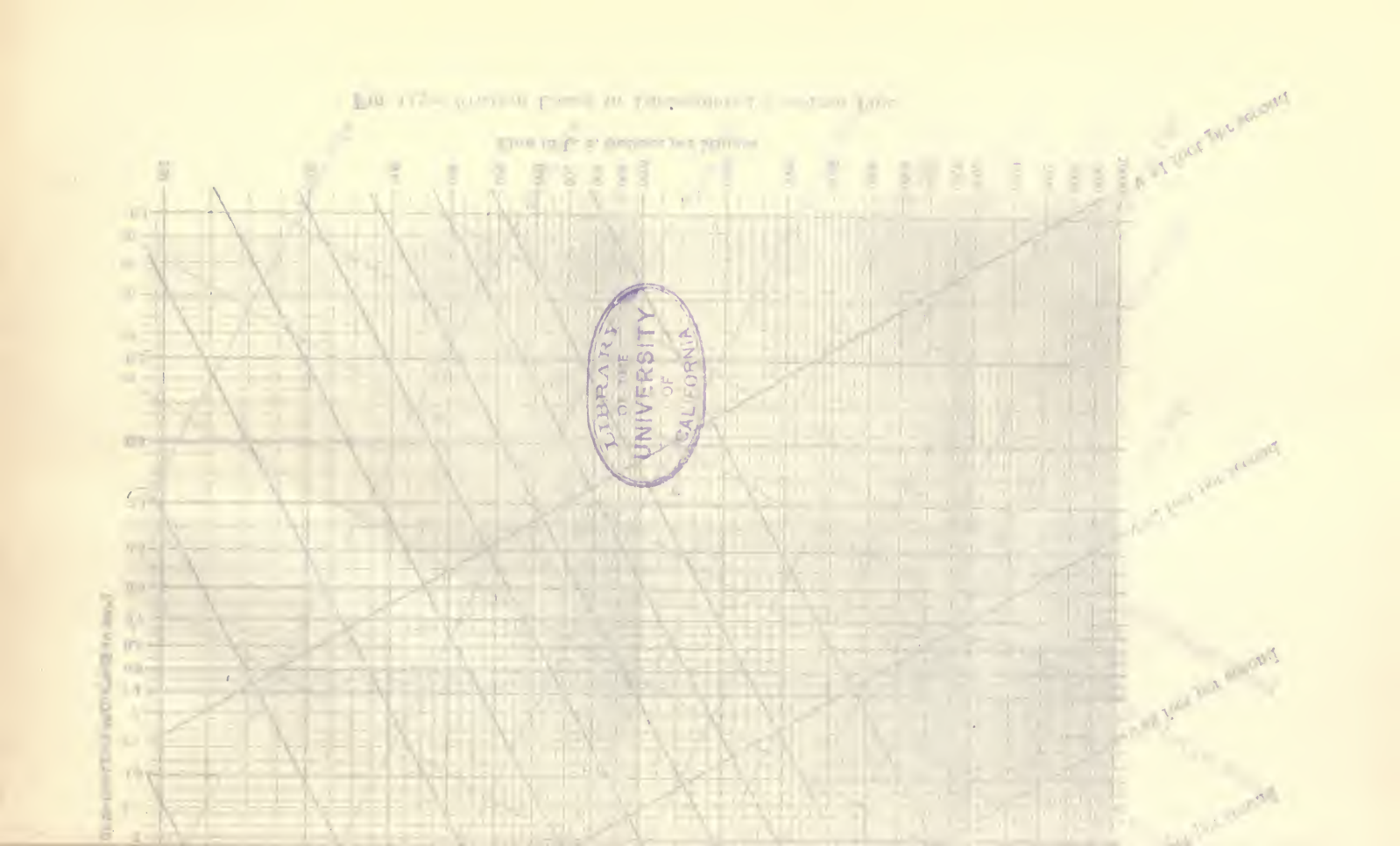
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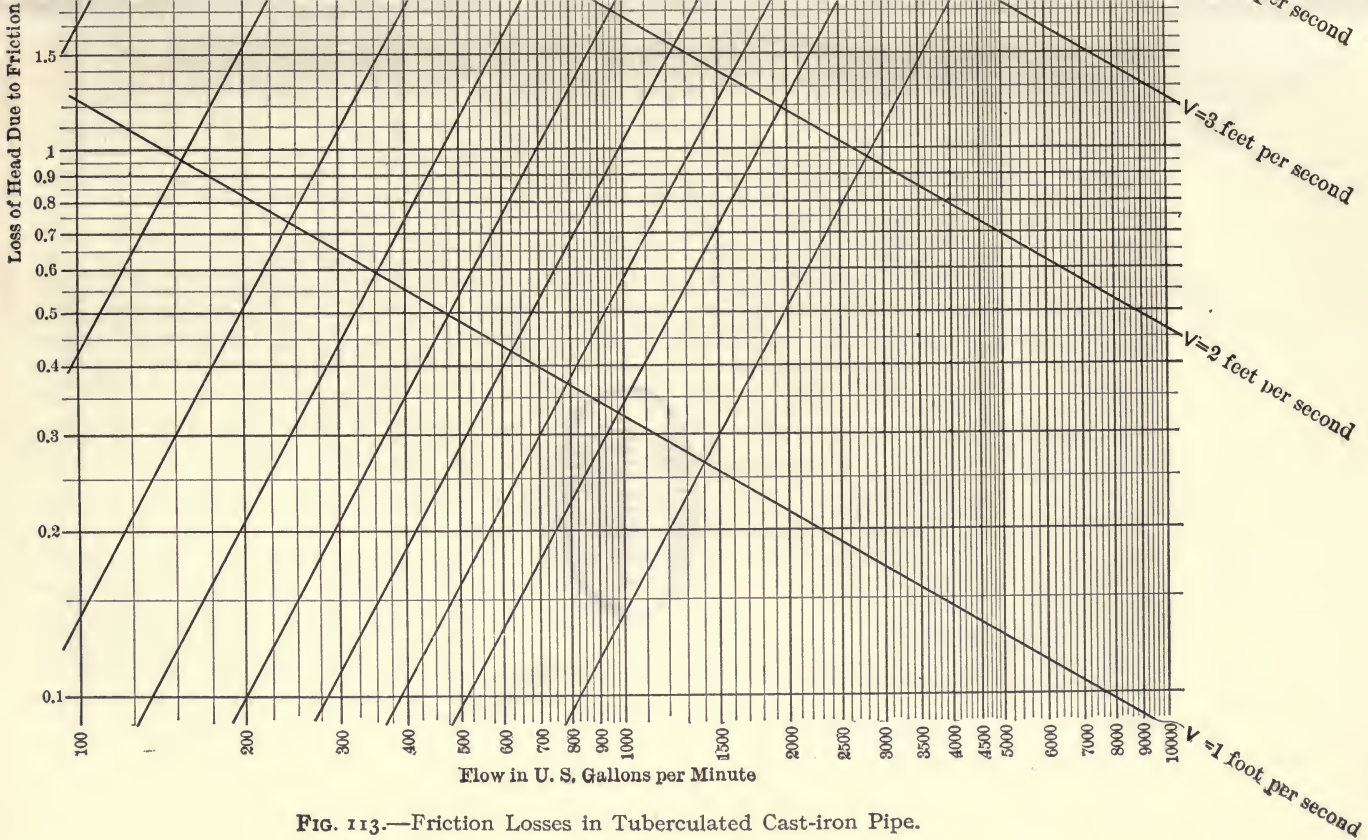
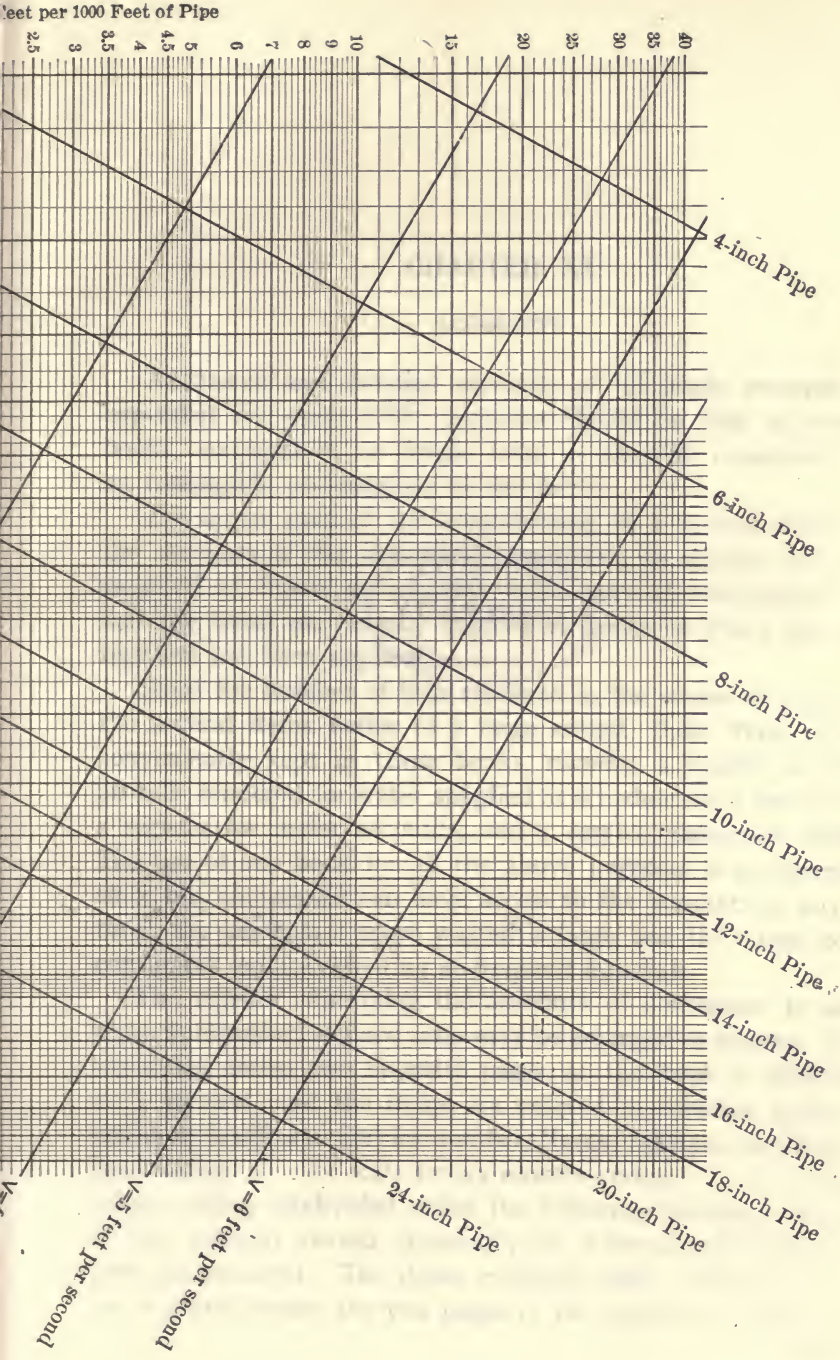
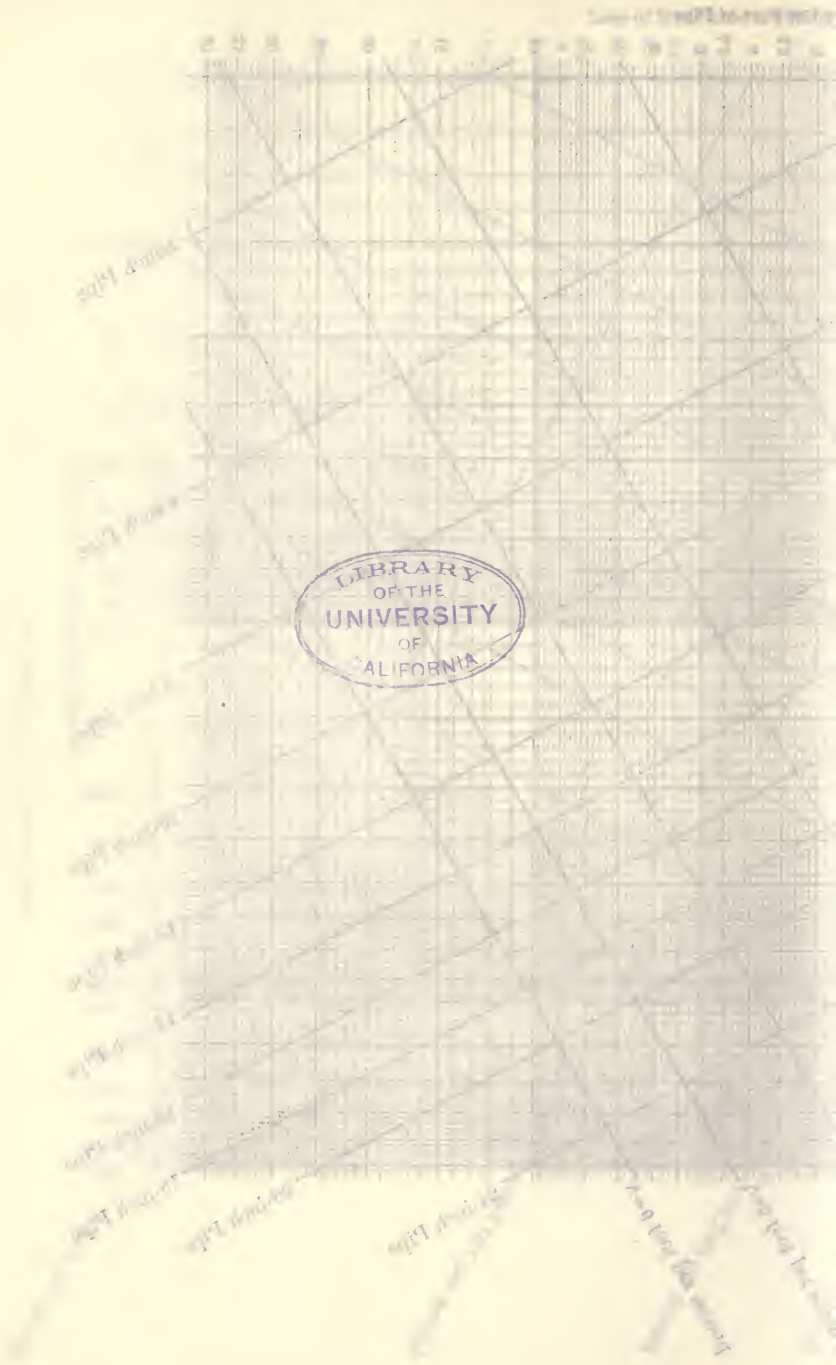


FIG. 113.—Friction Losses in Tuberculated Cast-iron Pipe.


Feet per 1000 Feet of Pipe
2.5
3
3.5
4
4.5
5
6
7
8
9
10
15
20
25
30
35
40
4-inch Pipe
6-inch Pipe
8-inch Pipe
10-inch Pipe
12-inch Pipe
14-inch Pipe
16-inch Pipe
18-inch Pipe
20-inch Pipe
24-inch Pipe
V=5 feet per second
V=6 feet per second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CHAPTER XI.

ACCOUNTS

ACCURATE and detailed accounts of all funds received and expended for water-works purposes should be kept in suitable books, arranged in as simple form as possible consistent with an intelligible presentation of the facts.

The larger part of the book-keeping of a system deals with the accounts of the individual consumers, as receipts for water supplied at meter or schedule rates, service connections, etc., and the items are usually entered in books to which the name registers has been applied.

Since the number of bills rendered in the course of a year for the several items varies to a large extent, these items may be conveniently kept in three books, namely, a register in which all bills rendered for water supplied at schedule rates are entered, a register for meter accounts, and a service-connection register. The use of one book for all the above purposes is inconvenient, since but one person can have access to the accounts at any one time; the amount of space wasted is large, and the names of the consumers require rewriting at frequent intervals.

The register containing the accounts of consumers to whom water is furnished at fixed rates may be arranged as follows: A line extending across two opposite pages of the book is allowed to each account, and the items are entered in columns under the following headings: Service number, Name, Address, and Rate for year ending .... (or Rate for six months ending ....), the latter column being subdivided under the following headings: Amount of bill, Amount abated, Amount paid, Amount uncollected, and Date of payment. The items included under "Rate for ...." are repeated across the two pages of the register in order that a

number of successive bills may be entered opposite the name of a consumer.

A similar arrangement may be used for the other registers by changing the column headings to conform to the purpose for which the book is intended.

Or the accounts may be kept by the card system, in which case a card is used for each consumer. Advantages derived from the use of cards for this purpose are: that several clerks may

April 1, 1904.

WATER RATE.

Service No. 45

John Smith

Amount $ 9.50

Abatement _____

Total $_____

Paid _____

Credit to
Water Maintenance Acct.

FIG. 114.—Coupon.

work on the accounts at one time, that changes due to the moving of consumers from one building to another, etc., may be adjusted more readily, and that the paid and unpaid accounts may be kept in separate files. Two disadvantages of the card system are that all figures must be taken off in computation books before total amounts can be ascertained, and the liability that cards may be misplaced or lost.

The bills are made from the data contained in the inspection, meter, and service records, and all bills rendered by the depart-

ment should have coupons attached upon which the date of bill, service number, name of consumer, amount of bill, etc., are noted. These coupons are torn from the bills by the person to whom payments are made, and returned to the water-works office. In order that the coupons of bills made for different purposes may be easily distinguished, the bills and attached coupons may be printed upon paper of different tints, or the type or the color of ink used on the coupons may be varied.

The amount of a bill is entered in the proper column of the register before the bill is sent out. Upon receipt of the coupon from the collecting officer the amount paid or abated is entered in the register. The total receipts on the different accounts as shown by the coupons are computed and the results entered in the cash-book or journal daily, weekly, or monthly, as may be found convenient. The coupons should be preserved until the accounts are made up for the year, and may then be destroyed.

At the close of the fiscal year the total footings of the columns of the registers should be ascertained. The aggregate amount of the bills rendered should equal the aggregate totals of the various subdivisions of the register, and the aggregate amount of the payments should equal the totals of the entries made in the cash-book or journal.

In general all bills rendered by the water department are payable within thirty days, but the instance is extremely rare where all bills are paid before the expiration of that period. Duplicate bills are occasionally rendered, but usually statements of accounts due, or notices so worded that the attention of the delinquents is called to the fact that bills are overdue, are sent to the consumers after a reasonable time has elapsed. In case the statement or notice does not accomplish its purpose, a shut-off notice is sent, and if this notice is disregarded the supply is discontinued until the arrears are paid.

The receipts from various sources and the expenditures incurred for construction, maintenance, and operation may be kept in a cash-book, journal, and ledger of the ordinary form. Or separate sets of books may be kept for the Construction and Maintenance accounts. Or in the case of small works where the number of detailed accounts kept is small, the entries may be

made in a single book under headings arranged on two pages somewhat as follows:

RECEIPTS.

			CONSTRUCTION.			MAINTENANCE.		
Date.	Received from:	Total amount.	Loans.	Services.	Sundry acc'ts.	Schedule rates.	Meter rates.	Sundry acc'ts.

EXPENDITURES.

			CONSTRUCTION.			
Date.	Paid to:	Total amount.	Distribution.	Services.	Meters.	Special.

MAINTENANCE.					
Administration.	Pumping.	Care of appurtenances.	Repairs.	Meters.	Sundry acc'ts.

The receipt or payment is entered in the column headed "Total amount," and also credited or charged to the proper account as indicated by the headings. The footings of the columns are carried forward from one page to the next, and the condition of the accounts, or any one account, can be readily determined when desired.

All bills, particularly those for supplies and material purchased, should be filed after payment for future reference.

In order that the data with regard to items of cost may be accurately compiled for the various records mentioned in Chap-

ter I, or for other purposes, both Force and Material accounts should be kept. These accounts are made daily and filed at the water-works office by the foremen or inspectors in charge of the various portions of the work. The form shown below is adapted for use with a small force working under the direction of a foreman or inspector. The blank forms are about $3\frac{1}{2}'' \times 6''$ and may be carried conveniently in the pocket in a loose-leaf binder or in pads. The names of the employés appear on the sheet, the work performed is noted in the space at the head of a column, and the time given by each employé is entered below

WATER-WORKS FORCE ACCOUNT.

Oct. 10, 1904.

	Leak 10" main, Pine Street.	Repairing hydrant No. 3, Main Street	Setting meter No. 4, Pine Street.	Service connection, W. H. Rice.	
John Williams, foreman...	5 hrs.	1 hr.	½ hr.	2½ hrs.	
Henry Murray.....	6 "	3 hrs.			
H. Kelley.....	6 "	3 "			
J. Smith.....	6 "		1 "	2 "	
Fred White.....	6 "			3 "	

opposite the name. The pay-rolls are made from the daily Force accounts. Daily reports of the amount of material used, the character of the material, and where used, should be made upon prepared forms or upon the back of the Force sheet.

In the case of continuous work, such as extensions of mains, etc., the time of the individual laborers may be more conveniently kept in a time-book, and the aggregate time entered on a Force sheet. This time may be apportioned to items such as teaming, excavation, pipe-laying, back-filling, etc., and data regarding the progress of the work, the material received, removed, or used, should also be given.

A Stock account is very desirable and may be kept in a book prepared for that purpose, in which the amount, kind, and cost of material purchased and used is entered. At the close of the year the figures should be checked by making an inventory of the stock on hand.

CHAPTER XII.

FINANCIAL MANAGEMENT.

To the requirements that extensions of, or additions to, a system of water-works be made in accordance with good engineering practice, that the efficiency of a system from a mechanical standpoint and the sanitary quality of the supply be maintained or improved, should be added the requirement that the department be operated on a business basis. In general, conditions favorable to such operation more often obtain in works owned and operated by private companies than when these are strictly municipal enterprises. Even in the latter instances, however, the affairs of the department may be conducted in a business-like manner if political or personal interests are eliminated.

At the outset a distinction should be made between funds expended and obtained for purposes of construction and of maintenance. The cost of the original works is charged to the Construction account, and expenditures for extensions, additions, or improvements are also properly chargeable to that account. Under this heading are included the cost of new mains and appurtenances, services, meters, pumping machinery, reservoirs or stand-pipes, filtration works, and expenditures incurred for the improvement or extension of the source of supply.

Expenditures for the purpose of renewing portions of the system, such as mains and appurtenances, pumping machinery, etc., which are either worn out or of insufficient capacity, are usually charged to the Construction account. Current practice is in this respect, however, varied. In some cases the cost of renewing minor portions of the works, as small mains, services, and meters, is charged to the Maintenance account or is divided between the two general accounts. In any case it is advisable

that the accounting of renewals be made in such manner that expenditures for original construction and for renewals may be distinguished from each other.

Funds for purposes of construction are usually provided by the issue of bonds or notes. When the works are owned by the municipality, these bonds or notes bear interest at certain specified rates, usually 3 or 4 per cent per annum, and are payable at a stated time, usually ten, twenty, or thirty years, after the date of issue. In some municipalities the cost of extensions is paid from assessments upon abutting property.

Minor items of construction, as yearly expenditures for new service connections or meters, are often, although charged to the Construction account, paid for from the surplus annual revenues.

Extensions of existing works which have as their object the improvement of the efficiency of the system are often made without regard to any financial benefit which would be directly derived therefrom. Such is not usually the case, however, when mains are extended in new streets, in streets not previously supplied with water, or in private ways. Mains are not generally laid in public streets unless the revenue to be received from the sale of water will yield a fair return upon the investment. When the revenue based on the established schedule of rates is insufficient for this purpose, and the indications are that sufficient revenue may be received in the near future, extensions are occasionally made, provided the water-takers served by the particular extension agree to pay, as special water rates, a sum equivalent to the fair return for a stated period of time. These special rates are fixed to produce an income of from 4 to 7 per cent annually on the cost of the work. Where extensions are made in private ways, the entire cost of the work is usually borne by the parties benefited, although provision may be made that the cost be refunded at such time as the ways become public.

The cost of service connections outside street limits is borne by the consumers. The cost of connections within street limits is in some cities and towns assumed by the department; in others a stated sum varying with the size of connection is charged. As a general rule, service connections are not laid unless the revenue

to be derived therefrom is at least equal to the established annual rate for a single faucet.

Water-meters should be owned and controlled by the water department, and not by the individual consumers. The cost of meters is ultimately borne by the consumers in either event, and meters may be removed for testing or repairs, or replaced, with less friction between the department and the consumers in the former instance than would be the case in the latter.

All expenditures incurred for the maintenance and operation of a system of water-works are charged to the Maintenance account. The items of expenditure included in this account are the salaries of administrative and executive officials, office expenses, wages of permanent employés, pumping expenses, purification expenses, care and repair of the system; in brief, all expenditures incurred in maintaining the system in good condition and for its operation. The annual interest charges on outstanding bonds or notes are also paid from the Maintenance account. In addition, payments to a sinking-fund which shall be sufficient to pay the outstanding indebtedness when it becomes due are made from this account, where the works are owned by the municipality. Where works are owned by private companies, similar annual payments may be made to a depreciation fund, as a provision for the replacement of portions of the system when this becomes necessary. In this latter instance a reasonable profit in addition to the foregoing charges is to be desired.

The funds required to provide for the varied expenditures for maintenance and operation are derived from the receipts from the sale of water, minor receipts for repairs and the sale of material, and from the general tax levy. These receipts constitute the Income account. The principal sources of revenue are the water rates and the appropriation from the tax levy, and the proportion of the annual income derived from the former, as compared with that derived from the latter, is subject to variation within wide limits in different communities.

Primarily the water rates represent the value of the water actually or presumably used by the individual consumers, and the appropriation in a measure represents the value of the fire protection and other indirect benefits afforded by the works.

The several departments of a municipality should be considered upon the same basis as private persons, and the water department should receive funds in the form of cash payments or credits for all water supplied for municipal purposes. This includes water used in public buildings, fountains, for sewer flushing, street sprinkling and cleaning.

Water rates are divided into two general classes: schedule, fixed, or assessed rates, and meter rates. The former are based, not upon the amount of water actually used, as are the latter, but upon the amount presumably used, the convenience or luxury derived from the practically unlimited use of the supply, the opportunity afforded for the waste of water, or a combination of these items. The methods by which the amount of the annual rate is ascertained in a given instance differ widely in practice. In general, schedule rates are based upon one or more of the following items: size of lot, street frontage, assessed value of building supplied, number of persons supplied, number of rooms, number and kind of fixtures, and annual rental value of premises. In the majority of works the rate annually assessed upon a building or apartment occupied by a single family is limited by a so-called maximum rate, which is usually from fifteen to twenty-five dollars.

Rates for metered water are either uniform, i.e., one price per unit of quantity regardless of the amount consumed, or vary with the quantity furnished. In this latter case a certain price per unit quantity is charged for the supply up to a definite amount, and reduced rates are charged for excess quantities. The following is an example of a graduated schedule of this kind:

For quantities not exceeding 3000 cubic feet per quarter, per 100 cubic feet..... $0.20
Excess above 3000 up to 10,000 cubic feet per quarter, per 100 cubic feet..... 0.15
Excess above 10,000 up to 50,000 cubic feet per quarter, per 100 cubic feet..... 0.10
Excess above 50,000 cubic feet per quarter, per 100 cubic feet..... 0.075

A graduated scale which offers any inducement for con-

sumers to waste water should never be established. A schedule of this character would read somewhat as follows:

For quantities not exceeding 3000 cubic feet per quarter, per 100 cubic feet..... $0.20
For quantities not exceeding 10,000 cubic feet per quarter, per 100 cubic feet..... 0.15
For quantities not exceeding 50,000 cubic feet per quarter, per 100 cubic feet..... 0.10
For quantities exceeding 50,000 cubic feet per quarter, per 100 cubic feet..... 0.075

Under a schedule of this form a consumer would, in cases when the consumption approached one of the limits given, be inclined to waste water in excess of that limit in order to realize the benefit of the lower rate.

Whenever water is sold by measure, a definite sum is fixed as the minimum annual rate regardless of the actual registration of the meter. The minimum rates established in different works range from about five to fifteen dollars per year. Two principal reasons for the establishment of a minimum rate are advanced. The first, which may be termed the financial reason, is that a certain amount of money is required each year to meet the fixed charges of maintenance and operation, including interest charges and sinking-fund requirements. The interest on outstanding bonds, sinking-fund payments, the salaries of permanent officials and employés, and the cost of repairs are affected but slightly, if at all, by variations in the total amount of water annually furnished. By the collection of minimum rates, funds applicable to the partial payment of these fixed charges are assured. The second reason is that as a sanitary measure excessive economy in the use of water should not be encouraged, and under the minimum rates in use an ample supply for an average family, if waste be restricted, may be obtained for all domestic purposes. Established rates for water supplied by meter measurement to domestic consumers range from twenty to thirty cents per thousand gallons, or about fifteen to twenty-five cents per one hundred cubic feet.

In an ideal system the annual revenue from rates and the appropriation should equal the annual expenditures plus the

contribution to the sinking- or depreciation-fund and the interest on the outstanding bonds or notes; the cost of the service should be equitably divided between the consumers and the taxpayers, and the rates should be equitably apportioned among the individual consumers. This result can, however, be only approximately realized under the most favorable conditions. Usually the annual receipts from the sale of water show an increase year by year, and the surplus revenue is used to make larger contributions to the sinking- or depreciation-fund. Or the amount of the hydrant rental or contribution from the general tax levy is diminished.

In addition to the direct benefits which the water-takers of a community receive from a system of water-works, the installation of works of this kind results in more or less indirect benefit to both consumers and taxpayers. In consequence of the existence of water-pipes in the public streets, real estate abutting on these streets is increased in value, the rental value of tenements and apartments supplied with water is likewise enhanced, and a saving in fire-insurance premiums is effected. The capacity of a system of water-works is increased, often to considerable extent, to provide for the large rates of flow necessary for efficient fire protection, and also as a provision for future demands, above the ordinary requirements of a community for domestic and manufacturing purposes. A certain proportion of the annual expenditures may, therefore, be equitably assessed upon taxable property. According to various authorities, the provisions for fire protection increase the cost of a system of works about one-third. The expenditures for maintenance and operation are likewise somewhat increased, hence it is assumed that 50 per cent of these expenditures, including interest and sinking-fund charges, may be taken as an approximate measure of the value of the fire protection and other indirect benefits afforded by the works, and may be properly raised by general taxation. The remaining 50 per cent of the annual expenses may then be apportioned among the individual consumers. The equitable division of this portion of the expenditures is best accomplished by the use of meters.

The method for the apportionment of the cost of maintain-

ing and operating a system of water-works outlined above is rarely followed in practice at present, particularly in the establishment of a schedule of water rates for new works. It is, however, of value in the adjustment of existing schedules. Rates established in accordance with the method outlined are based both upon the cost and the value of the service, and these two factors constitute the only proper and reasonable basis for such establishment. Since the cost of service is affected principally by the local conditions which obtain in a given case, and the interest charges and sinking- or depreciation-fund requirements depend upon the cost of construction, the cost of service may be widely different in any two communities. This difference is particularly marked if comparisons are made between works respectively supplied by gravity and by pumping. Rates established by the method indicated would, when based upon the cost of maintaining and operating a new system with a comparatively small number of consumers, be excessive and the increase in new business, in consequence, limited. Furthermore, the average consumer pays little or no attention to the cost of delivering water upon his premises, but bases his criticisms with regard to the reasonableness of rates upon comparisons made with schedules which obtain in adjacent cities or towns. It is the usual custom, therefore, when a supply of water is introduced in a city or town, to base the schedule of rates in large measure upon the schedules in use in works in close proximity to the case at hand. The yearly deficit is then made up by appropriations from the general tax levy, and during the first years of operation the payments to the sinking-fund are small. The appropriation from the general funds of a municipality for water-works purposes is usually made in the form of hydrant rental, i.e., a specified sum is paid for each public hydrant connected with the works.

Since funds are necessary at all seasons of the year for the payment of bills incurred by the department, it is customary to collect the whole or a portion of the water rates in advance. Bills for water furnished at schedule rates are usually rendered annually or semi-annually, and are payable in advance. Where water is furnished at such rates, the total annual revenue to be

received for a given year may be quite closely computed, and this fact constitutes the chief advantage of this method of assessment. On the contrary, the revenue to be derived from the sale of water by meter measurement is often very uncertain. The minimum rates may be collected in advance, but the total receipts can only be approximately ascertained until the close of the fiscal year. Meter bills are usually rendered quarterly or semi-annually to domestic consumers, and monthly or quarterly to manufacturers or other large consumers.

Water furnished for manufacturing purposes is usually metered, and the rates at which it is sold, when the domestic consumption is not metered, are, in general, arbitrarily fixed. With a view to the encouragement of manufacturing enterprises the meter rates, particularly when on a graduated scale, are often made purposely low. In such instances a change from the schedule to the meter system of furnishing water to domestic consumers results in a decrease in the annual revenue unless the schedule is revised. Owing to this fact, due consideration should be given to the cost of delivering water, when manufacturing rates are established, in order that radical changes in the schedule may be avoided when the domestic consumption is also metered.

The proposition is often advanced that when water is furnished by measurement all consumers should be treated alike and on an equal basis, regardless of the quantities consumed, i.e., the water should be furnished at a uniform rate per unit quantity. An adherence to a uniform meter rate would in many cases cause large consumers to obtain water from sources other than the public supply, to the financial disadvantage of the department. Such conditions may also obtain that the quantities used by a few consumers constitute a considerable proportion of the total consumption, and therefore a material reduction in the unit cost of delivering water results. Under a uniform schedule the small consumers would then receive the benefit of the reduced cost. In general, the adoption of a graduated scale is consistent with good business policy, and such schedules, if good judgment is exercised in their establishment, do not favor large consumers at the expense of the small or domestic consumers.

An established schedule of rates should not be changed unless the results obtained from a study of the financial and physical condition of the works, present and prospective, indicate the necessity for or advisability of such change. In lieu of a general reduction in rates, discounts are allowed in some works on all bills paid within a specified time after they are due. This discount is usually in the form of a percentage; as, ten per cent of the amount of the bill if the same is paid within ten days from date, and five per cent if paid within twenty days. If unforeseen events necessitate changes in the schedule, the changes may be more readily effected under a discount system than under a fixed-rate system. Although in all well-managed works the loss from non-payments of rates is insignificant in amount, more or less time is used and friction occasioned in the collection of overdue water rates, hence cash customers may very properly be given the benefits arising from the discount plan.

A change from the schedule to the meter basis of rate assessment, with the accompanying general introduction of meters, in an established system of works is usually productive of the following results:

First, an increase in the expenditures for clerical and repair work, inspection, and interest and sinking-fund charges.

Second, a decrease in the expenditures for fuel, and possibly to a slight extent in the other pumping expenses when water is pumped.

Third, a decrease in the revenue from consumers, or,

Fourth, a redistribution of the charges for water furnished individual consumers.

If the decrease indicated in the second item exceeds the increase due to the first, the change is advantageous to both department and consumers. Such, however, is not the result ordinarily obtained. However, the diminution in the annual consumption resulting from the introduction of meters, considered in a previous chapter, may be advantageous if expenditures for additional construction or for purification are materially reduced thereby. If the annual income is just sufficient to meet the charges for maintenance and operation, a reduction in the amount of revenue is not desirable, and the fixed charges cannot be met

by a uniform minimum rate charge unless that charge be excessive. As a means of providing revenue sufficient to meet the fixed charges the graduated minimum rate method used by Mr. Freeman C. Coffin, C.E., is of value. The minimum rates established by this method, and the quantity of water furnished to a consumer under these rates, vary with the kind of fixtures supplied from the metered service connection. The minimum meter rates established at Merrimac, Mass., afford an illustration of the use of a graduated minimum scale. In these works a minimum charge of six dollars per year is made where ordinary faucets only are supplied, eleven dollars per year for faucets and water-closets, fifteen dollars per year for faucets, water-closets, and bath-tubs, and twenty dollars per year for the above fixtures with hose, etc. This latter charge is the highest minimum charge made for water supplied to a single family. The minimum charges are payable quarterly in advance, and excess bills based upon the regular schedule are rendered at the end of each quarter.

Provision for the cost of maintaining meters is made in some works by the collection of meter rentals. An ordinary service-meter set in place costs from ten to sixteen dollars. The expense of maintaining these small meters is about as follows:

Interest on first cost at 4% per annum	$0.40 to $0.84	
Contribution to sinking- or depreciation-fund, 4% of cost	0.40 to 0.84	
Annual maintenance: reading, inspection, testing, repairs, and clerical work	1.00	1.00
Total annual maintenance	$1.80 to $2.68	

At the expiration of twenty years the sinking-fund will be sufficient to pay the original cost of the meter or to replace it, and the life of an average meter is usually assumed to be equal to that period. Since this life is, in general, dependent upon the character and amount of water passed through the meter, the duration of service of a meter may be more or less than twenty years. Within this limit the items of maintenance affected by the amount of water consumed through a metered service are those of repair and testing only, and since these are a small proportion of the total annual expense, the cost of maintaining meters may be equitably divided equally among the individual

consumers without regard to the amount of water furnished. The sum of two dollars per meter is, therefore, usually charged for the annual rental of an ordinary service-meter. This sum is ncreased with the increase in size of the meter in proportion to the additional interest, sinking-fund, and repair charges. A further charge is at times included in the rental of large meters to provide for the amount of water which may pass these meters without registration. This charge corresponds to the value of the possible unrecorded flow at the established rates.

The following charges may then be, and are usually, made when water is furnished through metered service connections:

First, an annual rental depending upon the size of meter which will meet the expense of its maintenance, and also provide for the under-registration of large meters at small rates of flow.

Second, a minimum annual rate which shall provide in a measure for the fixed charges for the maintenance and operation of the works, and which shall prevent undue economy in the use of water for domestic purposes.

Third, a graduated scale of rates for metered water.

CHAPTER XIII.

RULES AND REGULATIONS.

RULES and regulations governing the use of water are usually prescribed by the water commissioners where the works are owned by a municipality, and by the company when these are private enterprises. Such rules and regulations as are established, or are in use, should be explicit and should be impartially enforced. A limited number of rules which are available in printed form, and are strictly enforced, are of more value in the management of water-works than a large number of regulations inscribed in records and forgotten or rarely observed by the parties concerned.

The following directions and reservations are commonly included in the rules established by various works.

An application for a supply of water is required to be made upon a prepared form and signed by the owner of the property supplied or his agent.

In some works the material and labor required for making a service connection is furnished, and the connection made by the department from the street main to the stop- and waste-cock in the cellar of the applicant's building. In other works the pipe is furnished and laid with the necessary appurtenances by the department to the street line only, all work beyond the street line being performed by plumbers. Or the entire work is performed by licensed plumbers, although in some cases the street main is tapped by employés of the department. Where any portion of the work of making a service connection is performed by plumbers, permits are issued by the department covering each connection made, returns upon prepared forms of the service and fixtures connected therewith are required, and the work is inspected by

employés of the department before it is concealed. The size of pipe, and the material of the pipe and fittings to be used, are usually specified by the department.

The method of making service connections, and the charges to be borne by the applicant therefor, should be clearly stated in the rules and regulations.

When the entire service connection from the street main to the building supplied is laid by the department, plumbers or other unauthorized persons are not allowed to disturb in any manner the pipe or fittings placed by the department.

Notice of alterations in, or additions to, the fixtures originally supplied is required from the water-taker or the plumber doing the work. In some works no notice is required where alterations made consist merely in the replacement of a fixture by another of the same class.

Unnecessary waste of water is prohibited, and water-takers are required to maintain the interior piping and fixtures connected thereto in good condition and repair.

Consumers are not allowed to furnish water to persons not entitled to its use, nor to conceal the purposes for which water is used.

Where service connections are not metered, the use of hose for sprinkling street or lawns or for irrigating gardens is usually restricted. Regulations restricting the use of hose are more readily enforced if the hour or hours of the day during which such use is permissible are stated explicitly.

When meters are owned by the department and are located in buildings, consumers are usually held responsible for damage to the meter due to their negligence or to freezing. Consumers are also required to protect pipes and fixtures on their premises from freezing.

The times at which bills are rendered and are due are stated, and when several tenants are supplied through a single service-pipe the property-owner is held responsible for the water rates.

When water is furnished at schedule rates, discounts or abatements are usually allowed, provided that notice is given when premises are vacant in order that the water may be shut off if the department so desires, and that fixtures not in use may be sealed.

Officials or employés of the works are allowed to enter the premises of water-takers at all reasonable times to inspect the service, and to test, repair, or replace meters.

The supply of water may be interrupted in case it becomes necessary to shut off a main for repairs or other purposes. Previous notice of such interruption is usually given whenever possible.

The water is usually shut off from the premises of any taker who fails to comply with the rules and regulations or neglects to pay water rates after having been notified to do so. In these instances a charge is usually made for letting on the water after the cause for complaint has been removed.

The department usually reserves the right to place a meter upon a service and to charge for water supplied at meter rates.

When connections are desired for purposes of fire protection, detailed plans of the location of all pipes, valves, hydrants, etc., are required. All outlets from connections of this nature are sealed, the seals to remain unbroken except in case of fire or when water is used for tests, unless the connection is metered or provided with other devices for indicating a flow of water through the pipes. Connections between the fire-pipes and those in ordinary use are not allowed.

CHAPTER XIV.

ANNUAL REPORTS.

THE water commissioners of a city or town which is provided with a public water-works system annually present to the municipality a report regarding the system under their direction and control, in accordance with the provisions of legislative acts, municipal ordinances, or as a result of established custom. These annual reports should furnish information concerning the condition of the works, financial and otherwise, the amount and nature of the work performed during the period covered by the report, recommendations with regard to any action that the municipality should take, and facts which will enable the community to deal intelligently with the recommendations made.

A brief statement of the financial condition of the works is usually presented in the reports. This statement should show the cost of the works, the amount of the outstanding bonds or notes, the amount of the sinking-fund, and the net debt at the date the report is made. The work performed for purposes of construction should be briefly described, and if any portion of this work has been done by contract, information with regard to the nature of the work, the name of the firm or individual to whom the contract was awarded, the amount of the contract, and the progress made, is usually given. When competitive bids are received a summary of the bids may be included.

Statements with regard to the extensions of main pipes and services, and the additional hydrants, valves, meters, etc., placed during the year, may be made in tabular form, similar to the forms followed in the construction records. Alterations in or renewals of any parts of the existing works should be treated in similar manner. The tabulations may be supplemented by

such explanatory matter as it may appear necessary or advisable to append thereto. Data with regard to the total amount of pipe of the several diameters, the total number of valves, hydrants, service connections, and meters of the several patterns and sizes in use, are to be desired.

The recorded results of such observations as are made of the precipitation on the catchment area, the yield of this area, or the flow of streams, and the meteorological observations, should be presented in more or less detail. Data with regard to the consumption of water and the operation of pumping or purification plants should also be given.

The annual financial statements should be made with a view to the preservation of data of value or interest to the department and the community rather than to the gratification of the curiosity of a few inquisitive individuals. The sums received from various general sources and the expenditures for construction, maintenance, and operation should be stated in concise form. Financial statements covering pages of detail regarding the amounts paid various persons or corporations for labor, materials, etc., are of little value as compared with summarized statements of total expenditures for pumping, purification, repairs, maintenance of meters, hydrants, etc. Summarized statements of the cost of maintaining and operating water-works make the reports of value, not only to the municipality served by the works in question, but to the water departments of other cities and towns.

Water-works officials may derive much of benefit at times from the reports of other works, particularly if data upon different subjects can be compared upon a uniform basis. Means for such comparison are afforded by the use of established forms, and the form for statistics adopted by the New England Water-works Association is now used by a number of public water departments. This form is given on the following pages.

SUMMARY OF STATISTICS

FOR THE YEAR ENDING.....

In form recommended by the New England Water-works Association.

.....WATER-WORKS.

....

(City or Town.) (County.) (State.)

GENERAL STATISTICS.

Population by Census of 19 ,.....
Date of construction,.....
By whom owned,.....
Source of supply,.....
Mode of supply (whether gravity or pumping),.....

PUMPING STATISTICS.

1. Builders of Pumping Machinery, ...
2. Description of fuel used,
   - *a.* Kind,.....
   - *b.* Brand of coal,.....
   - *c.* Average price of coal per gross ton, delivered, $....
   - *d.* Percentage of ash,.....
   - *e.* Wood, price per cord, $.....
3. Coal consumed for the year,.....lbs.
4. [Pounds of wood consumed]÷3 = equivalent amount of coal,.....lbs.

4*a*. Amount of other fuel used,.....

5. Total equivalent coal consumed for the year= (3) + (4),.....lbs.
6. Total pumpage for the year,.....gallons { with / without } allowance for slip.
7. Average static head against which pumps work,.....feet.
8. Average dynamic head against which pumps work,.....feet.
9. Number of gallons pumped per pound of equivalent coal (5),.....
10. Duty = $\frac{\text{gallons pumped (6)} \times 8.34\text{(lbs.)} \times 100 \times \text{dynamic head(8)}}{\text{Total fuel consumed (5)}}$ = ...

Cost of pumping, figured on pumping-station expenses, viz., $.....

11. Per million gallons pumped, $.....
12. Per million gallons raised one foot (dynamic), $.....

FINANCIAL STATISTICS.

RECEIPTS.		EXPENDITURES.	
Balance brought forward:		*Water-works Maintenance:*	
(*a*) From ordinary (maintenance) receipts,	$.....	AA. Operation (management and repairs),	$.....
(*b*) From extraordinary receipts (bonds etc.),	.....	BB. Special:	
Total,	$.....	.....	
From Water Rates.		....., ...	
A. Fixture rates,	$....	CC. Total maintenance,	$.....
B. Meter rates,	$....	DD. Interest on bonds,	.....
C. Total from consumers,	$.....	(CC+DD),.....	
D. For hydrants,	$....	EE. Payment of bonds,	.....
E. For fountains,	....	FF. Sinking-fund,	.....
F. For street watering,	.....	*Water-works Construction:*	
G. For pub. b'ld'ngs,	.....	GG Extension of mains,	$.....
H. For miscel. uses,	.....	HH. Extension of services,	.....
I. Gen'l approp'n,	.....	II. Extension of meters,	.....
J. Total from munic. depts.,.....		JJ. Special:	
K. From tax levy,	.....	....	
L. From bond issue,	.....	....	
M. From other sources:		....	
.....,	$.....	KK. Total construction,	.....
.....,	$.....	LL. Unclassified expenses:	
		.....,	.....
		.....,	.....
		MM. Balance:	
		(*aa*) Ordinary,	$....
		(*bb*) Extraordinary,	....
		Total balance,	.....
N. Total,	$.....	N. Total,	$.....

Disposition of balance, ...

O. Net cost of works to date, ... $.....

P. Bonded debt at date, ... $.....

Q. Value of sinking-fund at date, . . . $.....

R. Average rate of interest,.....per cent.

STATISTICS OF CONSUMPTION OF WATER.

1. Estimated total population at date,.....
2. Estimated population on lines of pipe,.....
3. Estimated population supplied,.....
4. Total consumption for the year,.....gallons.
5. Passed through meters,.....gallons.
6. Percentage of consumption metered,.....
7. Average daily consumption,.....gallons.
8. Gallons per day to each inhabitant,.....
9. Gallons per day to each consumer,.....
10. Gallons per day to each tap,.....
11. Cost of supplying water, per million gallons, figured on total maintenance (item CC), $.....
12. Total cost of supplying water, per million gallons, figured on total maintenance + interest on bonds, $.....

STATISTICS RELATING TO DISTRIBUTION SYSTEM.

MAINS.

1. Kind of pipe,.....
2. Sizes, from.....inch to.....inch.
3. Extended.....feet during year.
4. Discontinued...feet during year.
5. Total now in use,.....miles.
6. Cost of repairs per mile, $.....
7. Number of leaks per mile,.....
8. Length of pipes less than 4 inches diam.,.....miles.
9. Number of hydrants added during year (public and private),
10. Number of hydrants (public and private) now in use,.....
11. Number of stop-gates added during year,.....
12. Number of stop-gates now in use, ...
13. Number of stop gates smaller than 4-inch,.....
14. Number of blow-offs,.....
15. Range of pressure on mains .....lbs.
    to.....lbs.

SERVICES.

16. Kind of pipe,.....
17. Sizes,.....
18. Extended.....feet.
19. Discontinued.....feet.
20. Total now in use,.....miles
21. Number of service-taps added during year.....
22. Number now in use,.....
23. Average length of service,..feet.
24. Average cost of service for the year, $.....
25. Number of meters added,.....
26. Number now in use,.....
27. Percentage of services metered .....
28. Percentage of receipts from metered water (B÷C),.....
29. Number of motors and elevators added,.....
30. Number now in use,.....

PART III.

FRANCHISE. WATER RATES. DEPRECIATION.

ISOLATED families obtain a wholesome water-supply in sufficient quantity for individual use by exercising ordinary caution to prevent contamination, and with a sufficient expenditure of money they may even possess the convenience of water under pressure in their houses and drainage facilities for removing waste to a safe distance from the premises. However, these conveniences for family use are rather expensive, and unless the proper skill is bestowed upon their design and construction and operation they may prove a source of considerable annoyance and inconvenience, to say nothing of a menace to health.

Small communities depend upon individual exertion with a considerable degree of safety and satisfaction, because, as a rule, the buildings are so generally isolated that co-operation in matters of water-supply and drainage is rather more than less expensive than individual effort.

Communities in which the population is sufficiently dense to render the hazard from fire collective and the preservation of the purity of the individual water-supply interdependent, and in which the rights and interests are clearly interwoven, find economy and security in co-operation. The larger the community the more imperative becomes the necessity for united action as a municipality. A community desiring the advantages which united action for the common good bestows often undertakes the development and distribution of a public water-supply as a purely municipal enterprise, and with the undertaking accepts all the attending responsibilities. These responsibilities increase in the direct proportion in which the increasing density of the pop-

ulation of the community itself and of the adjoining neighborhood multiplies the dangers of water-supply pollution, and they call for a corresponding increase of vigilance on the part of those in authority. But in the interest of the public welfare municipalities have the right to delegate these obligations to others, and for various reasons, in the exercise of these rights in the past, the municipality has granted a franchise to some individual or public-service corporation, usually for some specified period of years, to construct and operate water-works, with authority to collect compensation from the water-consumers at rates not exceeding those of a specified schedule. Usually the ordinance granting the franchise also embraces a contract for furnishing public fire protection and for a restricted use of the water for other public purposes at a specified rate of compensation. Thus the water service becomes largely a commodity embracing not only a tangible necessity of life but also a particular form of service ministering to the comforts and convenience of the community.

The inviting prospects of gain in the production and sale of this commodity readily attracted private capital, and little difficulty was experienced in the past in either granting or disposing of a water-works franchise to parties who were in a position to get the funds required for construction. The essential element which gave stability to the enterprise in the eyes of the investor was the fact that the franchise carried with it a contract binding the city to pay semiannually, usually during the life of the franchise, a specified amount for each and every hydrant installed for fire protection. This hydrant rent, being a fixed income, was readily set apart and pledged as interest for money used in the construction of the water-works and was secured by mortgage on the plant and franchise. The plan of the promoters was frequently to make the money thus available meet the cost of the original construction. Sometimes it was more, sometimes less, than was necessary for this purpose.

The operating company was usually stocked for a considerable amount, but the stock in many cases represented comparatively little or no original cash investment. Thus the stock possessed altogether a speculative value, depending upon the

character of management, the quality of the water service, and the degree of prosperity and growth of the community served by the public-service corporation. In the more prosperous cities a rapidly increasing business may have brought value to the stock and accordingly may have resulted in profit to the promoters. In the smaller cities and towns of comparatively slow growth this plan of operation was not so successful, for the business, often increasing slowly, could scarcely support the interest on the cost of the original construction and meet the ordinary expenses of operation and repairs. Accordingly, the stock possessed little or no initial value, but frequently came to represent an actual cash investment in proportion to the amount of money expended in the support and maintenance of the property during the period of insufficient income for these purposes. Sometimes these conditions were aggravated in a large measure by an inferior quality of the water or of the service furnished.

Reorganization of the public-service corporation was not uncommon during the life of the franchise, and usually some improvements and extensions followed in the wake of the reorganization. Finally, in the larger or more prosperous communities, the service became entitled to and received profitable patronage. But, as a rule, in the smaller communities, considerable portions of the franchise life usually elapsed before water-works property became wholly self-supporting. Considerable time, effort, and expense were generally required to build up a profitable patronage, and so soon as this patronage drew upon the plant for water service to the limit of its capacity it became necessary to enlarge, extend, and improve in order to maintain the service and to accommodate new business.

But important and expensive improvements are subject to much delay and postponement when the necessity for these becomes apparent during the later years of a limited franchise, for the reason that an investment which provides for future as well as for present requirements is often of doubtful expediency in view of the uncertainty arising in connection with a renewal of a franchise at the expiration of the original franchise. This uncertainty and the natural wariness of capital to seek investment under such circumstances financially cripples some public-

service corporation and serves to delay improvements even though a failure to make them promptly results in a deterioration of service to a greater or less degree. Public-service corporations which have pursued a policy of anticipating requirements in making improvements, regardless of franchise limitations as to time, appear to have suffered no serious impairment of their investment as the result of the venture, but rather to have fortified their position to ask for an extension of their franchise privileges. Nevertheless there appears some good basis for the claim frequently made that the indirect effect of a limited franchise for water-works is to create conditions which serve to deteriorate the water service and in a measure the physical property during the later years of its time limitation. A remedy for this alleged defect has been offered in the form of an indefinite or unlimited franchise wholly independent of any contract, providing for the fixing of the charges for water in some non-partisan manner and subject to termination through purchase by the municipality at a price for the physical property fixed by mutual agreement, arbitration, or condemnation proceedings, and exacting a suitable penalty for failure on the part of the public-service corporation to render good service and to furnish wholesome water in sufficient quantity—the central idea being to secure, on the one hand, acknowledgment on the part of the municipality of the existence of vested rights until the franchise shall have expired through the purchase of the property by the city in the manner previously suggested, and, on the other hand, a binding obligation upon the public-service corporation to provide wholesome water and good service.

The suggestion has also been offered that in order to attract private capital it may be necessary to show a certain degree of liberality in the application of the purchasing power of the municipality by limiting the application of this power to five-year intervals for a period of the first ten to fifteen years of a franchise and thereafter limiting it only to the interval of a year, for the reason that as a rule the first period of ten or fifteen years is the most trying and uncertain period for the public water-service corporation.

There seems to be considerable justice at times in the claim

which is made with regard to the effect of the limited franchise in postponing improvements to the property of the public water-service corporation, and there is undoubtedly some merit in the suggestion of a few years' delay in the exercise of the option to purchase in order to allow proper time and compensation for the expense usually required to establish a water-works property on a paying basis and to recover the equivalent of the depreciation of the property resulting from progressive deterioration, and possibly also from the decrease in the market value of the materials of construction. Deterioration of the physical property is unavoidable and should be represented by a corresponding charge against the earnings of the property and by setting apart in some manner a portion of the income of the business as compensation for the loss.

But provision in the income for depreciation resulting from a fall in the market prices of the materials may be altogether offset by enhancement of value at some other period of the life of the franchise resulting from a rise in market prices above those prevailing at the time of construction. This situation might very readily be met by making legitimate cost the basis of valuation for purchase or for rating that portion of the income required to take care of the actual investment in physical property were it not for the fact that the principle of the use of the market prices of material prevailing at the time of purchase seems to be regarded in many quarters as a fixed principle in the valuation of tangible or physical property. Assuming this principle to be correct for the purchaser, then the option to purchase at one-year intervals could be readily exercised in a year of low prevailing prices, but not so readily exercised in the same manner under an option to purchase at five-year intervals, but as the five-year option is intended to apply only for two or three terms while the business is being established, the suggestion is made that in all probability the period of years so covered is one during which a city would be the least likely to exercise the option of purchase or desire to do so, and that the practical effect of an indefinite franchise in the matter of option to purchase would be to exercise that option during the period of one-year intervals. Were this assumption correct it would follow

that the investment should be protected by a sinking-fund supported by the water rents against depreciation of that character, but it is open to the objection that a large discrepancy between the prices of material at the time of construction of a water-works property and at the time of purchase is more imaginary than real, for the reason that in the ordinary course of events either the original construction or the purchase of a water-works property is undertaken at a season of general prosperity when the prices of material are firm and in all probability above rather than below the general average of the market prices.

The situation is just this: the owner or seller of a water-works property bases his estimate of the value of his physical property at cost, and the purchaser of that property bases his estimate of value on the cost of duplicating the physical property at the time of purchase less depreciation; therefore it follows that the income from the sale of the commodity for which the plant was constructed must return the owner not only the interest up to the time of purchase, but also a sinking-fund covering the loss through depreciation of all kinds if his investment is secure. In the event of a failure of the income after deducting the necessary expenses of operation, repairs and interest to net a sufficient amount to provide the stated sinking-fund, the owner naturally feels that the value of the physical property computed on the basis of cost of reproduction less depreciation should be increased at least by an amount which would equitably compensate him for his losses during the preceding years, or that his ownership be undisturbed until losses thus incurred have been returned to him with interest through the earnings of subsequent years.

On the other hand a purchaser is quite likely to regard these losses as an incident of a somewhat speculative venture under a contract which not only fixes the rates which can be charged for water-service, but also allows the purchaser the option to purchase at stated intervals, and accordingly the purchaser is inclined to ignore any obligation to make good losses of the character described.

The conditions and circumstances which develop these radically different views have also developed quite as radical views with

regard to the interpretation of that portion of the franchise ordinance which relates to the valuation of a water-works property at the time of purchase—the owner maintaining that the business which he has accumulated and the unexpired portion of his franchise, as well as the physical property, are equally elements of value and should all be valued; the purchaser maintaining that the franchise is simply a grant which renders the physical property worth the cost of reproduction at the market price of the materials of which it is composed, less depreciation, and therefore possesses an inherent value only, or one which admits of no identity or valuation independent of the physical property.

It is a fact, however, that the owner of a water-works property is subjected to more or less pecuniary loss while he is accumulating a patronage and at the same time meeting regularly the fixed charges which are necessary to properly operate, support, and maintain his investment, and therefore may be entitled to a consideration of these losses in the valuation of his property, for the reason that the purchaser in taking possession of an established patronage as well as a physical property becomes possessor of a going concern with an established income and is thereby the gainer.

The purchaser, however, while he may concede the equity of considering a going value in connection with value of physical property, in so far as going value relates to the expense of accumulating a patronage that is sufficient to operate the physical property and support and maintain the *actual* investment therein, may not so readily concede the equity of considering going value as it relates to the establishing of a patronage and earnings in excess of those required for the above-stated purposes or those applied to the support of a fictitious investment, particularly if the purchaser is the grantor of the franchise which makes the conditions possible.

The franchises of the past are not comprehensive enough with regard either to the rules which should govern in appraising a water-works property or which should govern in the formation and preservation of an equitable schedule of water rates—in fact this weakness may extend to the statutory laws or even to the constitution of a State.

The rate of interest on a water-works investment is a matter which depends upon circumstances in general. The rate may be moderate or even low under an unlimited franchise if the principle of vested rights is generally acknowledged and morally accepted (for if morally accepted it can easily be made legally accepted) and if the water rates are fairly and non-partisanly adjusted to the amount and general character of the investment in physical property and to the necessary expenses incurred in establishing the business on a paying basis.

Of course the option of the municipality to purchase as applied to one-year intervals eliminates future earnings from consideration in the matter of valuation; and the power to adjust water rates periodically, to meet the requirements of each city in particular as occasion may require, eliminates speculative value from such property, and reduces the water-works problem to one of good engineering and sound business management. It may be difficult to finance a project shorn of its speculative attractions at the present time, particularly a private water-works project, for the shearing process having been applied indiscriminately in many instances during or at the expiration of limited water-works franchises, for which condition of affairs municipalities and public-service corporations are about equally responsible, needs some modification and the use of a methodical and equitable basis of valuation before mutual confidence is restored.

Evidently there is room for improvement of the conditions under which public water-service corporations operate—improvements which would be generally beneficial and should command the attention of the public and of legislators in some States at least.

Water as furnished in the form of a public water-supply is regarded largely as a commodity. The products of the farm and industrial institutions are also commodities, but they are sold by specific measurement, while water as a commodity is sold often without any pretense of measurement. Water is cleverly regarded as a product of nature and to that extent may be properly considered free to all. So is air a natural product, but were air

to be cooled or warmed artificially, conducted in pipes, and circulated in dwellings for the seasonal comfort of the inmates the service thus rendered would possess a value as a commodity which should not be wasted through carelessness or extravagance, and so with solar heat—it is free to all as naturally dispensed, but turned into energy artificially and disposed of as a commodity, the service thus rendered would have value in definite proportion to the amount consumed. Even coal, the natural product of solar heat, possesses comparatively little value until mined and transported for use, when it is sold as a commodity by definite measurement.

But with water service too little consideration is given to the expense of collecting and distributing it, and of removing the pollution naturally or artificially present, to arouse unqualified assent to treating it thoroughly as a commodity subject to sale in a definite amount. The sale of water service without measurement for a specified annual cost, regardless of the amount of water used, is manifestly a crude basis of dispensing a commodity, accommodating itself to a day and an age when a cheap and convenient method of measurement was unknown. It is a method which appeals to the sense of justice neither of the seller nor the purchaser, but is nevertheless strongly intrenched behind custom.

The value of a specific volume of any commodity depends, first, upon the costs of producing and distributing it; second, upon the cost of maintaining indefinitely the methods and means through which the production and distribution of the commodity are effected. All of these costs become distinctive elements of value which vary relatively in different localities. The greater the distance water must be conveyed, the higher the elevation to which it must be lifted, and the more labor, fuel, and supplies that are consumed in collecting, purifying, and distributing water at serviceable pressure, the greater becomes the cost and the value of the water service. It may require 20 feet of pipeline in one city to carry water from one consumer to the next consumer, and 100 feet in another city; but one thousand gallons of water in one city is one thousand gallons in another city, and it may cost five cents to make that one thousand gallons

serviceable to a consumer in one city and ten cents or more in another city. Therefore the water-service tariff must be graded to suit different conditions and localities if it is expected to produce an income which will properly support the investment in water-works property in all cases. A deficit in this particular may affect a public-service corporation more directly than it affects a municipality, because of the latter's ability to make up any deficit of water-service revenue through a tax levied upon the property of the citizens. But even the taxing power of a city is limited by law, and if a deficit in water-service income continues and finally becomes too great annually to be made up through special taxation, a floating debt accumulates which may become a serious matter, whether the municipality can or cannot absorb it legally through an issue of bonds. A situation of this kind is likely to confront a city engaged in the management of public utilities, particularly if expenditures in construction are heavy: either the water tariff to the consumer must be proportionately heavy or the tax upon the property more or less burdensome. Under such circumstances the city cannot permit water to be wasted and should sell above the cost of production in cases where the property is taxed for interest and sinking-fund of bonds issued for construction purposes. The only safe course for small cities in general and those in particular heavily indebted for water-works is to sell water service by measurement and above cost, yet not at a price which is prohibitive. For a year or two in the beginning of the public water service, while connections are being made for consumers, a deficit of revenue may result, which may have to be provided for either through a special tax or out of the general fund until made up out of the surplus revenue of succeeding years.

It is difficult indeed to present figures convincingly to a community on the eve of embarking upon a water-works management and to impress upon it the importance of adopting a water rate fitted to its local requirements or of insisting at the outstart on adopting methods of management which will insure good plumbing and freedom from waste and extravagant use of water. As a rule, the community regarding water service cheaply adopts the annual rate sheet of some neighboring town, possibly reduc-

ing the rates somewhat, regardless of its adaptation to the local situation and requirements. The fact that the water-works do not pay, but annually accumulate a deficit, is something of a revelation and becomes the source of more or less political bickering. It may require several years of experience to locate the source of trouble and still longer to correct it, for the difficulty politically of raising a schedule of water rates is great even though the necessity for doing so is apparent to every one.

The error committed is a double one: first, in ever considering for a moment any other method of selling the water than by actual measurement; second, in not having made the rate high enough to cover the cost of producing the water service, after allowing one to three years for acquiring patronage, or in not making it even higher, for if made too high it can be gradually reduced without difficulty after making up the first year or two of loss.

However, there is a form of water service which must be considered, but to which the observations of the few preceding pages relating to the water service for domestic and trade purposes must be applied with considerable modification. Probably the annual rate for the use of an indefinite amount of water is the most practical and convenient method of charging for water consumed in fire-service, excepting so far as the fire-hydrants may be used for other purposes than the extinguishment of fires, when the actual volume so abstracted from a hydrant should become the basis of sale and purchase. This modification becomes necessary largely because there is no simple and satisfactory way of measuring the large volume of water used for fire purposes. The danger of waste of water in fire-service is small, for ordinarily the penalties imposed by city ordinance for the unauthorized use of fire-hydrants are sufficiently rigid to prevent waste of water, in view of the fact that the hydrants are upon public ground and continually exposed to public gaze and police surveillance. Moreover, the water actually used for fire-service annually is small in the aggregate, but large relatively for the brief periods that it is required for use.

The essential principle of good fire protection is a prompt and capacious service which can be applied in large volume imme-

diately upon the discovery of a fire and before it can have time to spread to uncontrollable proportions. The application of this principle requires, first, an abundance of water and a capacity of plant in excess of that required for the ordinary daily consumption; second, a continuity of service that can respond efficiently to fire requirements upon demand. In the densely occupied districts of the business centers of large cities, where there may be many large and lofty buildings, the demand for adequate fire protection is so excessive as often to require a system of street pipes and pumping machinery entirely independent of the system used for the distribution of water for general purposes. This kind of protection any city possessing building districts of the kind described, particularly if occupying a site of rugged topography, can well afford to provide, and the city should impose a direct tax upon the property immediately benefited, covering the cost of operation and possibly of maintenance of the district fire protection, thus freeing the city as a whole from all obligation except that of providing the means for making the original investment in physical property for this specific purpose.

The expense of fire-service for which property, generally speaking, is liable is at least twofold: one the expense of supporting and maintaining the investment in the physical property in excess of that required for the ordinary water-service consumption; the other the expense of the annual outlay required in support of an ever-ready and prompt fire-service. It is clearly the part of the property to bear this expense.

The public-service corporation in its dealings with the municipality has attempted to meet the situation by charging the city some specific amount annually for each hydrant as hydrant rent and obligating the city to provide the hydrant rents by annually levying a tax for the purpose upon property. The governing principle underlying the annual charge for fire-service, termed hydrant rent, was intended in many cases to produce a distinct fund of an amount which in the aggregate would be sufficient to pay the interest on the cost of the original water-works. The application of the principle of property participation in water-works maintenance in the manner described, though it does

not discriminate between an investment made for fire protection only and that made for ordinary industrial and domestic purposes, is in effect but little different from the case when a municipality building its own water-works taxes the property for the interest on the bonds issued for the purpose of general construction and for the support of a sinking-fund to redeem the bonds at maturity. The difference in the two applications of the same principle is solely the difference of the rate of interest paid on private and city bonds respectively, in view of the fact that the obligation to support a sinking-fund should rest equally upon both parties. Neither application of the principle may be strictly equitable as between the obligations of the property and the consumer in the support of a water service; still, in spite of its shortcoming, it may have been and may yet be under existing conditions an easy practical way of solving a problem otherwise intricate and uncertain, at least for the five or ten years following the first construction of a water-works. After a period of years, which may vary in length in different localities, the principle may be applied upon a more equitable basis; that is to say, it may be based upon an equitable division of the investment as between the requirements for fire-service and those for domestic and industrial purposes.

But in view of the fact that hydrant rent has developed into a stumbling-block for both the public-service corporation and the municipality, it seems as though further discrimination should be made in proportion to the benefits received and to the fire hazard to which property is exposed in different districts of a city. It is known that a densely built-up portion of a city is more exposed to a fire hazard than is the residential portion, and that this hazard further decreases in proportion to the degree of isolation of independent residences. In a city of rugged topography, buildings on low ground receive better fire-service than those on high ground, and buildings near are better served than buildings remote from the center of water distribution. The mechanical facilities for stopping a fire are more accessible and more promptly concentrated in the business portion than in the residential portion of a city, a situation which affects only indirectly the available service of the pipe system. Moreover, the

pipe capacity, often carried through low-lying and near-by districts in order to reach high-level and remote districts, affords the low-level districts a direct benefit which is not available to less favorably located districts. It would seem, therefore, that hydrant rent should be graduated by a sliding scale in proportion to the efficiency of the available service, depending upon location geographically and topographically with respect to the densely built-up populated district and the center of water distribution. The argument may be advanced that all districts of the city should be treated alike in this regard and a flat-rate hydrant rent prevail, for the reason that a wide-spread calamity in a business district extends its effect throughout the city regardless of boundaries artificially established, and that consequently all are equally benefited morally and socially in the facilities and preparation for avoiding such a calamity. But the argument cannot apply to a property-holder in both the business and residential districts, for his pecuniary liability is balanced in the application of a sliding-scale system of fire-protection charges, and should not apply to the wage-earner depending upon the progress and integrity of the business district, for generally he is not a direct participant in the profits of the business or in the rents of real estate.

A sliding scale of hydrant rentals should embrace scarcely more than three or four classifications of hydrants, but of an amount which should be fixed for each town independently, depending upon the natural conditions affecting the distribution of water and the rendering of a proper fire-service.

The custom ordinarily followed in the past of introducing in a franchise ordinance the height of some stated number of hose-streams delivered through specified length of hose and size of nozzles to some stated height, as a basis of measuring the efficiency of a proper fire-service, is rather a crude method of measurement and subject to a conflicting interpretation of results. Its only merit is its impressiveness in facilitating opinions which may be either favorable or unfavorable, depending upon the visible result, regardless of the effect of the hose between the hydrant and the nozzle upon the height and efficiency of the stream issuing from the nozzle.

The resistance of fire-hose to the flow of water, though it is a matter usually well understood and appreciated by a public-service corporation, is often misunderstood or disregarded by the casual observer and even by members of a fire department. Controversies over fire-stream efficiency on particular occasions have arisen, and the most frivolous sort of evidence has been introduced in court in support of a claim of inefficiency of fire-service, which a resort to the development of the facts and circumstances could have avoided.

The painstaking and elaborate experiments of John R. Freeman have given us a reliable standard with which the efficiency and capacity of fire-hose may be very satisfactorily compared and deficiencies observed. Commercial considerations, often controlling the selection of an equipment of a fire department, may result in an inefficiency of fire-service for which the system of street pipes and its equipment is in nowise responsible. Observations recently made of fire-department hose gave the results expressed in the following table:

TABLE I.

Equipment.		Pressure measurements.			Friction loss from Freeman's tables.	
Length of hose in feet.	Size of nozzle in inches.	Nozzle in pounds.	Hydrant in pounds.	Actual resistance per 100 feet in pounds.	Inferior rubber-lined cotton-mill hose, inside rough.	Ordinary best quality rubber-lined hose, inside smooth.
100	1 S.*	100	124	24	44	25
100	1 S.	82	107	25	37	22
100	1 S.	66	87	21	30	16
100	1 S.	62	85	23	28	15
100	1 S.	60	80	20	27	15
100	1 S.	45	60	15	20	11
500	1 S.	34	91	11.4	13.6	7

* 1″ smooth nozzle.

The tests, with the exception of the first one recorded, show a deterioration of hose or predominating tendency to a selection of rather an inferior quality of hose, although in the assortment there were some sections of a very good quality. The first test under 100 pounds pressure indicates a friction loss of a less amount than that indicated by Freeman's tables, a circumstance which

was probably due to the expansion of the hose while under the stated high working pressure.

To illustrate the effect of the variation of the diameter of hose from the nominal diameter of $2\frac{1}{2}$ inches the following computation is made under the recognized physical law that the discharge varies as the square foot of the fifth power of the diameter, and the resistance inversely as the fifth power of the diameter, hence the following tabulation:

TABLE II.

Diameter of hose in inches.	Ratio of resistance.	Ratio of discharge.
$2\frac{3}{8}$	1.29	0.88
$2\frac{7}{16}$	1.13	0.94
$2\frac{1}{2}$	1.00	1.00
$2\frac{9}{16}$	0.88	1.06
$2\frac{5}{8}$	0.78	1.13

The table shows that for a decrease of hose diameter of $\frac{1}{16}$ of an inch the resistance is increased 13 per cent or more, and the discharge decreases 6 per cent, and for an increase of hose diameter by a like amount the resistance is decreased 12 per cent, and the discharge increased 6 per cent.

The illustration develops good reasons for desiring a better basis of determining the efficiency of fire-service relating to the street pipes than a physical test of fire streams. The fact that Freeman's tables are accepted as a standard affords opportunity of basing measurements of fire-service efficiency of the pipe system upon pressure observations at the hydrant, leaving the responsibility for the use of inferior grades of hose upon the party responsible for its selection, where it belongs.

The progressive introduction of meters is gradually placing water-works upon a more rational basis of dispensing water service as a commodity. But notwithstanding the fact of progress in this direction there is still a well-marked popular objection to the use of meters, which only time and acquaintance with this system of measurement can overcome. However, in view of the past prevalence of the annual rate system, it is interesting to see its effect upon revenue and the financial support which it has afforded water-works property.

The Fourteenth Annual Report of the Commissioner of Labor for 1899 gives information from which the following data have been compiled. In the computations no distinction is made between revenue earned by meter and that earned by the application of the annual rate system, but generally the annual rate predominates. A showing is made of gross income independent of the hydrant rent or the income from the sale of water used for public purposes, in order that the income of private water-works may be compared with the income from municipal water-works. This comparison may not be altogether a fair one, for the reason that the extent to which the taxing power of municipalities may have been used to make up a deficit of income from water rate is unknown.

TABLE III.—PRIVATE WATER-WORKS.

Average annual consumption in million gallons.	Average annual income and cost per million gallons.				
	Gross income.	Gross income, less hydrant rent.	Total cost of production and taxes.	Gross income, less cost of production.	Interest on investment.
1 to 5	$447.60	$275.50	$195.28	$252.32	$2.90
5 " 10	347.60	226.11	159.58	188.02	3.50
10 " 15	252.10	167.24	162.74	89.36	2.80
15 " 20	237.20	141.35	118.16	119.04	5.06
20 " 25	216.40	169.05	106.94	109.46	5.42
25 " 50	152.40	112.50	65.37	87.03	5.24
50 " 75	128.11	98.48	50.43	77.68	6.65
75 " 100	118.30	87.04	60.02	58.28	4.71
100 " 125	97.30	66.91	43.00	54.30	6.04
125 " 150	105.90	78.39	47.79	58.11	5.26
150 " 175	90.20	63.80	45.84	44.36	5.63
175 " 200	98.10	67.71	55.51	42.59	4.85
200 " 250	96.30	73.28	41.98	54.32	6.33
250 " 500	70.50	55.22	29.40	41.10	6.36
500 " 750	58.90	44.30	26.93	31.97	5.06
750 " 1000	61.80	48.50	26.33	35.47	5.55
1000 " 5000	56.30	46.42	21.93	34.37	5.53
5000 " 10000	113.60	100.74	39.93	73.67	7.10

Examination of Tables III and IV shows clearly that, notwithstanding the comparatively large gross income received and the reasonable cost of production annually per one million gallons by private water companies, the net income averages only a small rate of interest on the investment, if provision be made for a sinking-fund. The reason of this in small communities

is due in a great measure to the fact that the cost of the pipe system, comprising the greater portion of the expense of the water-works plant, is relatively far greater than the large pipe system of a large town when compared upon a basis of capacity. For instance, a 48-inch pipe line will cost about four times as much as a 12-inch pipe line, and the latter about three times as much as a 4-inch pipe line; but the capacity to furnish water at an equal velocity of the 48-inch pipe is sixteen times that of the 12-inch pipe, and the latter nine times that of the 4-inch pipe, while the capacity for an equal friction loss per unit of length is, similarly, for the 48-inch thirty-three times the 12-inch, and the latter nearly eighteen times the 4-inch pipe capacity. Accordingly, small communities are frequently more heavily taxed per unit of pipe capacity for the construction and maintenance of water-works than are large communities.

TABLE IV.

Average annual consumption in million gallons.	Municipal plants.		Private plants.
	Average annual income and cost per million gallons.		
	Gross income.	Cost of production.	Cost of production, less taxes.
1 to 5	$203.10	$298.80	$146.40
5 " 10	157.90	150.30	153.50
10 " 15	144.50	115.80	138.20
15 " 20	109.00	101.80	96.20
20 " 25	110.80	84.80	102.00
25 " 50	84.00	60.60	58.50
50 " 75	74.30	42.30	43.40
75 " 100	79.20	46.10	51.10
100 " 125	63.90	34.20	36.80
125 " 150	64.00	38.10	37.70
150 " 175	58.00	16.40	40.80
175 " 200	69.30	25.40	47.00
200 " 250	88.90	26.90	36.30
250 " 500	61.50	22.70	25.10
500 " 750	70.80	25.20	20.60
750 " 1000	61.00	19.50	19.40
1000 " 5000	59.30	17.50	17.60
5000 " 10000	47.10	10.70	29.10
Over 10000	52.60	16.70	

The following interesting comparisons may be made of the preceding tables, omitting consideration of the first three groups

embracing annual consumption of water from 1 to 15 million gallons:

Average total gross earnings of private plants, per million gallons	$113.42
	Per cent of total gross earnings.
Average gross income private plant, less hydrant rent	73.7
Average gross income private plant, less cost of production	54.2
Average cost of production and taxes	45.8
Average cost of production, less taxes	38.9
Average gross income of municipal plant	63.6
Average cost of production of municipal plant	32.5
Average rate of interest which gross earnings, less taxes and cost of production, will pay on private investment is shown to be	5.65%

Deductions made from the foregoing table show the excess income of the private water-works over municipal water-works, amounting to $41.28, to be distributed as follows per one million gallons:

Average hydrant rents	$29.83
Average taxes	7.83
Income from commercial sources	3.62
	$41.28

That portion of the total gross income which remains after deducting the cost of production and taxes is seen to pay an average of 5.65 per cent on the average investment. Many of the public-service corporations mortgaged the water-works property to raise the funds for construction, and, from sources of information independent of the report referred to, it is found that out of a total of 358 private water-works in 27 different States scattered throughout the Union 61.2 per cent floated securities at 6 per cent, and that the average rate of interest of all the securities is 5.82 per cent.

It is also noticed that the cost of production of the municipal plants is $7.26 per million gallons less than the corresponding cost, exclusive of hydrant rent and taxes, of private plants—a difference due largely to the greater amounts expended in salaries under private management. The discrepancy is not material except in relation to the larger plants, where it is quite as likely to be due to conservatism on the one hand as to liberality on the other. The discrepancies in three groups of plants

would indicate such a probability and that an average might consistently be taken of the cost of production of those groups of plants where the difference is material. The predominating character of the system of water-works, whether gravity or pumping system, would necessarily influence the average in the respective cases.

Were the discrepancy to be averaged it would make the average rate of interest on the average investment scarcely 6 per cent. It is apparent, therefore, that the statistics show in a general way little or no speculative value in private water-works property as a class, also that the average income has been insufficient to provide a sinking-fund to cover depreciation resulting from the progressive deterioration of the physical property during the usual life of a limited franchise.

This condition of income of water-works operated under a limited franchise must sooner or later react disastrously and represents a substantial foundation for the claim now frequently heard of the importance of the unlimited franchise as applied to the successful operation and maintenance of this particular form of public-service property. It is also clear that a 4 or 5 per cent investment under an unlimited franchise giving good service would be worth more to an investor and would prove more satisfactory to the consumers than a 6 per cent investment under a twenty-year franchise with every incentive offered at times to squeeze the service in order to protect the investor from loss through unavoidable depreciation and the uncertainty of the future when the franchise shall have expired.

It is quite certain that the influence of metered services instead of the predominating influence of the annual rate system during the past experience of water-works public-service corporations would have acted as a protection to the water-service patrons and the investor jointly by eliminating an incentive to economize unduly and eliminating also the losses attending the waste of water and service which has prevailed to a greater or less extent under the influence of the annual rate system.

So long as water service is considered a commodity the income from the sale of it under private management should pay the interest on the investment in physical property actually required

to produce the commodity, a sinking-fund to redeem the investment eventually and to renew the physical property as it becomes useless, operating expenses, minor repairs, taxes, insurance, and in addition a reasonable surplus to meet emergencies. The longer and more secure the franchise under which a private property is built and operated the lower should be the rate of interest and sinking-fund.

A municipality operating its own water-works as though producing a commodity must necessarily follow a similar course, except that the interest and sinking-fund in part may be, as they usually are, a tax upon the property, and the income from the consumers may be limited to the actual cost of production plus a part of the sinking-fund and an excess to cover emergency expenses and repairs. But a city is privileged to cease treating the water service as a commodity, or to deliver the water at cost or below, and to provide for a deficit in any amount by apportionment from the general fund of the city, or by a special or general tax upon the property of the city, or any combination of these facilities that may be convenient and legal. The departure from the commodity basis is not liable to result in any economy or improvement of the water service to the consumer or the public generally, and is rather a dangerous expedient for small towns to adopt, as pointed out on preceding pages. The method will prove a fitting companion to the annual rate basis of dispensing a water service, for the reason that it places no check upon extravagance or waste, and presents no lesson in the matter of either personal or civic economy. Moreover the burden of the expense attending either the reckless or extravagant use of the water service falls, as a rule, most heavily upon the provident rather than the improvident, upon the careful rather than the careless or extravagant citizens.

Among the objections to the general use of a water-meter, the prejudice or misunderstanding of the general public is more easily overcome than is the objection of the owners of rental property who desire to avoid responsibility for the careless use of fixtures by tenants. Experience will usually overcome prejudice in this regard, but some method may be necessary to compel the owner of rental property to make repairs which he per-

sistently neglects or will not make voluntarily. It is clear as a matter of self-protection that the tenant will insist upon freedom from waste through leaky fixtures, but has no power to compel the owner to remedy the defect. The right to shut off the water should be exercised with discretion in such a case and need not be exercised at all if the owner, after due notice, fails to make the repairs, provided the value of the water service so wasted can be charged against the property and legally become a first lien against it. A law or ordinance of this character or a similar one would prove effective.

The method of dispensing a water service by definite volume must finally prevail, and with a view of facilitating the computation of a gross income a table is prepared, based upon a general average but somewhat modified cost of producing the water service contained in Table IV. The computation also embraces separately the elements of interest and sinking-fund when this support is to be provided.

The computations are intended simply as a guide to the judgment, and the cost of production is regarded simply as a base rate subject to modification in particular cases, as indicated in the memoranda immediately following the table, in instances where more definite data are not available.

The cost of production is supposed to embrace all of the charges, such as salaries of administration and operation, wages, insurance, fuel, supplies, and other minor items of cost.

The average working pressure of the works embraced in the computations of Table V is 60 pounds, and fully 85 per cent of the works have pumping machinery.

The average salaries of administration and operation are 21 per cent, and average wages 26 per cent of the total cost of production, between the limits of 100 and 1000 million gallons annual consumption.

Taxes should be added to the cost of production given in Table V whenever they become an element of the annual expense of operation.

It does not follow from an inspection of the table that a flat meter rate is suggested, but that in each particular case, after the cost of production shall have been modified to suit the locality

and after such additions to cover interest and depreciation of value of the physical property shall have been made, the final

TABLE V.—BASE RATE PER MILLION GALLONS.

Average annual consumption in million gallons.	Average investment per million gallons annual consumption.	Average cost of production.	Cost of production plus interest on average investment at—				
			4%	5%	6%	7%	8%
1 to 15	$4692	$146	$334	$381	$427	$474	$521
15 " 25	2168	96	183	204	226	248	269
25 " 50	1663	59	125	142	159	175	192
50 " 100	1209	46	94	106	119	131	143
100 " 150	1010	37	77	88	98	108	118
150 " 250	858	33	67	76	84	93	102
250 " 500	645	25	51	57	64	70	77
500 " 750	632	23	48	55	61	67	74
750 " 1000	640	20	46	52	58	65	71
1000 " 5000	622	18	43	49	55	62	68
5000 " 10000	600	17	41	47	53	59	65
10000 and over	568	16	39	44	50	56	61

The cost of production of—

Gravity plants 150 to 10,000 million gallons and over is 80 to 60 per cent of stated cost of production.

Pumping-plants 1000 to over 10,000 million gallons of high-grade engines pumping water once, as tabulated.

Pumping-plants 1000 to over 10,000 million gallons of high-grade engines pumping water twice, add 40 per cent to tabulated cost of production.

Pumping-plants 100 to 750 million gallons of medium-grade engines pumping water twice, add 40 per cent to tabulated cost of production.

TABLE VI.—AVERAGE COST OF FUEL FROM REPORT OF COMMISSIONER OF LABOR.

Annual consumption in million gallons.	Cost of coal per ton.	Annual consumption in million gallons.	Cost of coal per ton.
1 to 5	$2.14	150 to 175	$1.59
5 " 10	2.59	175 " 200	2.15
10 " 15	2.51	200 " 250	2.46
15 " 20	2.36	250 " 500	2.27
20 " 25	2.52	500 " 750	2.32
25 " 50	2.22	750 " 1000	2.09
50 " 75	2.36	1000 " 5000	2.26
75 " 100	2.16	5000 " 10000	2.64
100 " 125	2.02	10,000 and over	2.29
125 " 150	2.66		

result should be reduced to a sliding scale of cost per 1000 gallons of water, for application as a tariff upon the water service

of individual consumers in proportion to the amount of water consumed.

The matter of depreciation resulting from the deterioration of physical property should be considered in relation to plants under municipal ownership and administration as well as those under private ownership and control. Unless proper provision is made for the maintenance of the property under either kind of ownership and administration, that is to say, provision for the losses incident to physical deterioration, there must necessarily be a progressive shrinkage of value, for which there is no tangible equivalent. Provision of this character must come from some definite source, when the physical property subject to deterioration is assembled into a mechanical unit for a definite purpose, as for the production and sale of a commodity. Evidently the income from the sale of that commodity must be the source from which is derived the revenue with which to operate and maintain the physical property. If it is necessary to expend money in addition to that expended in the construction of the physical property, in order to make a market for the commodity in such amount and at such a price as to secure an income sufficient to operate and maintain the physical property and pay interest upon the investment therein, the amount of money so expended is a cost of establishing the business. Income eventually derived from the sale of the commodity which is in excess of that needed to support the physical property in the manner indicated and to pay interest upon the money expended in establishing the business is an asset against which negotiable paper of some sort may be and usually is issued in a purely private and unrestricted undertaking in some definite amount, which possesses value in the commercial market of an amount depending upon the interest or dividends which the excess income will support. Frequently a portion of this excess income is first set aside as a surplus fund available for emergency expenses or as a substitute for the capitalization represented by the negotiable paper and the remainder only applied to dividends.

In the case of a public-service corporation, depending upon a franchise for the right to construct and operate works for the

production and sale of a commodity at some fixed rate, the capitalization of the excess income as before described is sometimes termed franchise value. However, under a limited franchise containing express provision of the grantor's right to purchase and an implied intention of purchase at some stated interval or at the expiration of the franchise, it is often asserted that securities representing purely capitalized income should have but a limited circulation in the financial market at any time, as they may have no value at the expiration of the franchise or an indefinite value at a time of purchase before the expiration of the franchise. It is also maintained that the public as a grantor of a franchise receiving no compensation for the gift should not be expected much less required to buy back the franchise at the time of the purchase, particularly at the expiration of the period covered in the grant. Claim is also made, with regard to the rates under which water is dispensed as a commodity by a public water-service corporation, that they were never intended to accumulate more income than is necessary to pay operating expenses and to provide a reasonable fund for interest upon and maintenance of the investment, and that any income in excess of this should be considered the result of the use of an excessive water rate, and accordingly should be applied to the liquidation of the investment, or the water rates should be proportionately reduced.

The fact is when water-works franchises were granted years ago little thought was expended upon matters of this kind, the municipality desired the water service, and private individuals were ready to accept the obligation of supplying municipalities and to speculate somewhat upon the venture. What income the water rates would return was largely speculative, aside from the fixed income of hydrant rent. The question of franchise value as applied to water-service corporations becomes therefore a question of the moment and of the occasion, susceptible of consideration in some instances and of none in others, depending largely upon the terms of the franchise ordinance itself and the legality of and equity in claims of this character.

It is perfectly evident that the proper support and maintenance of the physical property is of first consideration and that there can be no consideration of franchise value whatever,

assuming the validity of such a claim to be open to consideration, until adequate provision shall have been made from the income for the support and maintenance of physical property.

It is clear that if a water-works property is appraised at a valuation based upon the cost of reproduction less depreciation, there should be more than presumptive evidence that the income has been sufficient to reimburse the seller for the losses which this method of valuation entails, as well as a fair interest upon the investment. Information regarding original cost and income and current expenses is not always available, and in the absence of proper and substantial information of this character approximations only can be made, based upon general information of a similar character.

Although only the depreciation resulting from deterioration of the physical property has been mentioned, the depreciation resulting from a gradual deterioration of service is also open to consideration. But a marked distinction should be made in the two types of depreciation. The former type of depreciation results solely from physical deterioration, the latter type relates to the earnings and represents depreciation of the business or "going value" of the water-works property. The one relates to elements entering the construction of the water-works, the other relates to elements of business management in dispensing the water service and in providing the necessary means for the proper dispensation of this service. While the deterioration of service may at times result from an incapacity of the physical property, still the measure of that deterioration can only be made through its depreciating effect upon the earnings resulting from the operation of the plant. In illustration of this principle a water-works plant which through the lapse of years develops an incapacity or deficiency in part or as a whole, which is manifest through imperfect or inadequate service, is subject to a charge upon its earnings over and above operating expenses of an amount which will pay interest upon and maintain the investment required in reinforcing the deficient part or parts of the plant.

The earnings are also similarly chargeable with an amount which will support properly the investment in and the operation of purification works needed to purify water from a source of

supply which, though originally producing an acceptable and wholesome water, has through the lapse of years become polluted through unpreventable sources or influences, or water from a supply which originally required purification, but which is inadequately purified through neglect to extend the purification works as needed, or through an effort to reduce operating expenses at the expense of the water service.

The converse of this situation is also true. The earnings of a business which does not properly support the original investment, together with that needed from time to time to reinforce the plant and to maintain an efficient service, should receive in themselves reinforcement through an increase, if necessary, of the schedule of charges for the water service.

Considerations of this character are particularly necessary when, for any reason, an effort is made to reach a commercial value of a property through a capitalization of net earnings. A neglect to observe them is particularly liable to result in overvaluation of a property appraised at or near the expiration of a limited franchise, and may in other instances result in undervaluation, when the actuating policy of the management is to reinforce and build in anticipation of future requirements.

Attempts are made to harmonize both types of deterioration by basing estimates of depreciation upon the serviceable life of the materials of construction as assembled in units rather than upon the actual physical life of the materials, independent of the useful life of the respective units into which they may be assembled. The serviceable life is shorter than the physical life in amounts depending upon the actual conditions and requirements of service.

There are three methods of computing depreciation which have had more or less extended application—each method recognizing progressive physical deterioration.

The first method assumes a certain annual rate of depreciation, and computes the annual depreciation for each year upon the original investment. For instance, assume a rate of depreciation of 2 per cent per annum—which is equivalent to an assumed physical life of fifty years—then the annual depreciation for each $1000 of investment by this method becomes $20 and for twenty years is $400.

This method of computation takes no account whatever of the interest earning capacity of the $20 annuities for each $1000 of investment during the interval for which the depreciation is estimated. If these $20 annual payments were considered as an annuity, drawing interest at the rate of 2 per cent per annum, it would amount to $1692 for each $1000 of the original investment at the end of fifty years. Accordingly an apportionment annually of $20 per $1000 investment from the gross earnings, and likewise the water rate which support such an annual apportionment of funds, becomes excessive.

If depreciation is estimated on this basis, it should be considered that the annual payments are immediately applied as partial payments on the original investment and the interest-bearing portion thereof accordingly reduced. For instance, assume the $1000 valuation upon which the depreciation has been computed to draw 6 per cent interest, then the increment by which the annual interest would be decreased, assuming the $20 annual payments to be immediately applied to payments on the original principal, is 6 per cent of $20 or $1.20, and if this latter amount should be considered as an annuity, it would have to be invested at about 8 per cent to amount to the difference between the $1692, the amount of the $20 annuity, and $1000, the original investment, or at a proportionately higher rate of interest for a 3 per cent investment, etc. Evidently this method calls for too heavy a rate of depreciation, and for an application of the annual payments not usually made.

The second method of computing depreciation assumes some rate of depreciation, as in the preceding case, but computes the depreciation on the principal of the preceding year instead of on the original investment continuously. For instance, the assumed rate of depreciation of 2 per cent a year would run as follows per $1000 investment:

			Unpaid principal.
End of first	year	$1000.00×0.02=$20.00	$980.00
" " second	"	980.00×0.02= 19.60	960.40
" " third	"	960.40×0.02= 19.21	941.19
" " twentieth	"	681.23×0.02= 13.62	667.61
" " fiftieth	"	371.60×0.02= 7.43	364.17

We find, therefore, at the end of twenty years the depreciation is \$1000.00 − \$667.61 = \$332.39, instead of \$400.00 by the preceding method.

The principle upon which this method of computation is based is that the present value of a property at the end of any interval is that sum which if the interest on it at some selected rate be compounded annually, it will at the end of the chosen interval of time equal the original investment. Theoretically, therefore, there must always be some present value to an investment in perishable property, and as the depreciation is computed as the difference between this value and the original investment it can never equal the investment. Effort has been made to overcome the apparent inconsistency of this method of computing depreciation by making computations upon the sinking-fund basis.

The third method of computation upon a sinking-fund basis assumes a life for the physical property, computes the annuity which at some stated rate of interest will equal the investment, and then the amount of this annuity for that portion of the expired physical life which may be under consideration. For instance, assume the physical life to be fifty years and rate of interest 2 per cent, then \$11.82 is found to be the annuity for each \$1000 of investment and the amount of this annuity for an expired interval of twenty of the fifty years becomes \$287 per \$1000, the amount of the depreciation, or, if considered on a 3 per cent basis, the depreciation becomes \$239, or \$197 on a 4 per cent basis.

This method contemplates refunding the investment at the end of some specific period, the duration of which depends upon the rate of deterioration of the physical property under consideration and an investment of the money set apart for this purpose in productive property or securities. The method is similar to that provided by law for the extinguishment of municipal, state, and national debts, and is being extensively employed in the valuation of water-works properties.

In order to apply this method of estimating depreciation, a sinking-fund table (pp. 414, 415) is given showing the annuity required to redeem one dollar at rates of interest varying from

TABLE VII.—SINKING-FUND TABLE FOR 1 TO 100 YEARS AT STATED RATES OF INTEREST.

Year.	Rates of interest in per cent.				
	2%	2½%	3%	3½%	4%
1	$1.00000	$1.00000	$1.00000	$1.00000	$1.00000
2	0.49505	0.49382	0.49261	0.49140	0.49020
3	0.32675	0.32514	0.32353	0.32193	0.32035
4	0.24262	0.24082	0.23902	0.23725	0.23550
5	0.19218	0.19025	0.18835	0.18648	0.18463
6	0.15853	0.15655	0.15451	0.15267	0.15077
7	0.13451	0.13250	0.13051	0.12854	0.12661
8	0.11651	0.11458	0.11246	0.11048	0.10853
9	0.10252	0.10046	0.09843	0.09644	0.09449
10	0.09133	0.08926	0.08723	0.08524	0.08329
11	0.08218	0.08011	0.07808	0.07609	0.07411
12	0.07456	0.07250	0.07047	0.06848	0.06655
13	0.06812	0.06605	0.06403	0.06205	0.06015
14	0.06260	0.06054	0.05853	0,05657	0.05467
15	0.05782	0.05577	0.05380	0.05183	0.04994
16	0.05365	0.05160	0.04961	0.04768	0.04582
17	0.04997	0.04793	0.04595	0.04404	0.04220
18	0.04675	0.04470	0.04271	0.04082	0.03899
19	0.04378	0.04176	0.03981	0.03794	0.03614
20	0.04116	0.03915	0.03722	0.03536	0.03356
21	0.03878	0.03678	0.03487	0.03304	0.03128
22	0.03663	0.03465	0.03270	0.03094	0.02920
23	0.03467	0.03270	0.03081	0.02902	0.02731
24	0.03287	0.03091	0.02905	0.02727	0.02560
25	0.03122	0.02928	0.02743	0.02567	0.02401
26	0.02970	0.02777	0.02594	0.02421	0.02257
27	0.02821	0.02640	0.02460	0.02285	0.02124
28	0.02699	0.02509	0.02329	0.02161	0.02000
29	0.02578	0.02389	0.02211	0.02045	0.01888
30	0.02465	0.02278	0.02102	0.01937	0.01783
31	0.02360	0.02174	0.02000	0.01837	0.01686
32	0.02261	0.02077	0.01905	0.01744	0.01595
33	0.02169	0.01986	0.01816	0.01657	0.01511
34	0.02082	0.01901	0.01732	0.01576	0.01431
35	0.02000	0.01821	0.01654	0.01499	0.01358
36	0.01923	0.01745	0.01580	0.01426	0.01288
37	0.01851	0.01674	0.01511	0.01361	0.01224
38	0.01781	0.01607	0.01450	0.01298	0.01163
39	0.01717	0.01544	0.01383	0.01240	0.01106
40	0.01655	0.01484	0.01326	0.01183	0.01052
41	0.01597	0.01427	0.01271	0.01130	0.01002
42	0.01542	0.01373	0.01219	0.01080	0.00954
43	0.01489	0.01322	0.01170	0.01033	0.00909
44	0.01439	0.01273	0.01123	0.00987	0.00866
45	0.01391	0.01226	0.01080	0.00945	0.00826
46	0.01345	0.01183	0.01036	0.00905	0.00788
47	0.01302	0.01141	0.00996	0.00866	0.00752
48	0.01260	0.01097	0.00957	0.00830	0.00718
49	0.01220	0.01066	0.00918	0.00796	0.00686
50	0.01182	0.01026	0.00886	0.00763	0.00655

TABLE VII.—SINKING-FUND TABLE FOR 1 TO 100 YEARS AT STATED RATES OF INTEREST.—*Continued.*

Year.	Rates of interest in per cent.				
	2%	2½%	3%	3½%	4%
51	$0.01146	$0.00991	$0.00853	$0.00732	$0.00626
52	0.01111	0.00957	0.00822	0.00703	0.00598
53	0.01077	0.00925	0.00791	0.00674	0.00572
54	0.01045	0.00894	0.00763	0.00647	0.00547
55	0.01014	0 00866	0.00735	0.00622	0.00523
56	0.00984	0.00837	0.00709	0.00597	0.00501
57	0.00956	0.00810	0.00683	0 00573	0.00479
58	0.00929	0.00784	0.00659	0.00550	0.00458
59	0.00902	0.00759	0.00635	0.00529	0.00439
60	0.00877	0.00735	0.00613	0.00507	0.00420
61	0.00852	0.00712	0.00592	0.00489	0.00402
62	0.00828	0.00690	0.00571	0.00471	0.00385
63	0.00806	0.00669	0.00552	0.00453	0.00369
64	0.00784	0.00648	0.00533	0.00435	0 00353
65	0 00763	0.00628	0.00515	0.00418	0.00339
66	0.00742	0.00609	0.00497	0.00403	0.00325
67	0.00722	0.00591	0.00480	0.00388	0.00311
68	0.00704	0.00573	0.00465	0.00373	0.00298
69	0.00685	0.00556	0.00449	0.00359	0.00286
70	0.00667	0. 00540	0.00434	0.00346	0. 00274
71	0.00649	0.00524	0.00419	0.00333	0.00263
72	0.00633	0.00508	0.00405	0.00321	0.00252
73	0.00616	0.00493	0.00392	0.00309	0.00242
74	0.00601	0.00479	0.00379	0.00298	0.00232
75	0.00585	0.00465	0.00367	0.00287	0.00223
76	0.00571	0.00452	0.00355	0.00277	0.00214
77	0.00557	0.00439	0.00343	0.00266	0.00205
78	0.00542	0.00426	0.00333	0.00257	0.00197
79	0.00529	0.00414	0.00321	0.00247	0.00189
80	0.00516	0.00403	0.00311	0.00239	0.00181
81	0.00503	0.00391	0.00301	0.00230	0.00174
82	0.00491	0.00380	0.00292	0.00222	0.00167
83	0.00479	0.00369	0.00282	0.00214	0.00160
84	0.00468	0.00359	0.00273	0.00206	0.00154
85	0.00456	0.00349	0.00264	0.00199	0.00148
86	0.00445	0.00340	0.00256	0.00192	0.00142
87	0.00434	0.00330	0.00248	0.00185	0.00136
88	0.00424	0.00321	0.00240	0.00178	0.00131
89	0.00414	0.00312	0.00233	0.00172	0.00126
90	0.00405	0.00304	0.00225	0.00166	0.00121
91	0.00395	0.00295	0.00219	0.00160	0.00116
92	0.00386	0.00287	0.00212	0.00154	0.00111
93	0.00377	0.00280	0.00205	0.00149	0.00107
94	0.00368	0.00271	0.00199	0.00144	0.00103
95	0.00360	0.00265	0.00193	0.00138	0.00099
96	0.00351	0.00258	0.00187	0.00134	0.00095
97	0.00343	0.00251	0.00181	0.00129	0.00091
98	0.00336	0.00244	0.00175	0.00125	0.00087
99	0.00328	0.00237	0.00170	0.00121	0.00084
100	0.00320	0.00231	0.00165	0.00116	0.00081

2 to 4 per cent and for an interval of one to one hundred years, the deposit being made at the end of each year.

The amount of an annuity of one dollar for any particular period and rate of interest is the reciprocal of the corresponding amount in the table. Accordingly in finding depreciation for a period of twenty years, for instance, on a 2 per cent basis, of property possessing a life of fifty years, divide the annuity which will redeem the investment in fifty years by 0.0412—which per $1000 is $11.82 ÷ 0.0412 = $287.

The rate of interest at which annuities to a sinking-fund may be invested should vary inversely as the life of the article of which the depreciation is sought, or approximately as follows:

For a life of				100	years	2	per cent
"	"	"	"	80	"	$2\frac{1}{2}$	"
"	"	"	"	60	"	$2\frac{3}{4}$	"
"	"	"	"	40	"	3	"
"	"	"	"	30	"	$3\frac{1}{4}$	"
"	"	"	"	20	"	$3\frac{1}{2}$	"
"	"	"	"	15	"	$3\frac{3}{4}$	"
"	"	"	"	10	"	4	"

In order to show the application of Table VII, the experience of a small city may be offered as an example:

Original cost of physical property	$56000.00
Capacity in million gallons per year	37
Average investment per million gallons annual consumption	$1540.00
Average working pressure	80 pounds

All water pumped twice with the ordinary grade of pumping-engines and filtered.

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1. Average cost of production of a water service of 37 million gallons per year, Table V	$59.00
2. Add for double pumping and filtering	24.00
3. Add for station expenses, exclusive of salaries required to pump against 20 pounds excess pressure	10.00
4. Add $1\frac{1}{2}$ per cent of the average investment per million gallons annual consumption for surplus to meet emergency expenses and to provide for renewals	23.00
Total, exclusive of interest, taxes, and sinking-fund	$116.00

The estimated cost of 11.6 cents per one thousand gallons would usually be exceeded during the period that the town referred to is gradually approaching the rate of consumption of 37 million gallons per year and similarly reduced as the annual consumption exceeds the 37 million gallons.

Tables III, IV, and V may be employed as an aid to municipalities desiring to rate the water service of a new water-works or to rerate the water service of a recently purchased water-works. An actual case of a water-works plant owned and operated by a municipality is offered to illustrate the distribution of revenue from the sale of water service.

Average annual consumption in million gallons	385.40
Investment per million gallons annual consumption	$1200.00
Cost of fuel per ton	4.00
Average pressure in city	80 pounds
Pumping-engine duty in million foot-pounds	33

	Per million gallons.
The actual cost of production, including the necessary expenditure for maintaining the service	$59.00
The amount of interest at 4 per cent paid on the existing balance of the original cost of construction	26.00
The amount applied to sinking-fund to redeem the bonds representing the unpaid cost of construction is about	9.00
The amount applied to reinforcement and similar current expenses	23.00
Gross income	$117.00

The city has been forty years in paying off about 50 per cent of the cost of constructing the water-works, and during the interval has maintained the plant generally in an efficient condition, but deterioration of the source of water-supply has reached such a stage that purification works are now required at an estimated cost of construction of nearly $60,000, without prospects of any corresponding increase of earning capacity of the water-works, but with a proportionate increase of the cost of operation and of the current expenses of maintenance.

This example illustrates both a successful management and the advisability of maintaining water rates considerably above the cost of production, in order to provide for emergencies of service and reinforcements as the necessity arises from time to

time. The example affords additional illustration of a city treating the water service on a commodity basis and accordingly charging the income from consumers for interest and sinking-fund, and at the same time charging the city annually over $6000 for the public use of the water service.

Assuming the city to have purchased the property at its cost of $1200 per million gallons annual consumption, and desiring to rerate the water service through the use of the table, it might proceed as follows:

	Per million gallons.
Average cost of production, Table V.	$25.00
Add cost of fuel above average cost and of maintaining the pressure of water service above average and a small percentage of double pumping, $25 \times 53\% \times 1\frac{1}{2}$	20.00
	$45.00
Interest on investment at	4.00%
Sinking-fund at 3 per cent to redeem the investment in about 50 years, as per Table VII.	0.90
Contingent expenses of operation and maintenance	1.5
Total per cent.	6.40%

Thence by Table V opposite $1209 investment under 6 per cent column take 119, deduct the corresponding tabulated cost of production of $46, multiply by $\frac{6.4}{6}$, add $45, the cost of production estimated above equals $123 per million gallons as a fair base rate for the stated conditions, which base rate should be so apportioned among the consumers as to produce a gross annual income of about $47,400. But, in the event of the construction of a filter, the expenditure before stated of $60,000 must be supported by the earnings to the extent of 6.4 per cent on the investment for interest, sinking-fund, etc., and operating expenses provided for to the amount of about $2 per million gallons, aggregating about $12 per million gallons, making the corresponding base rate $135 per million gallons, and the gross annual income a little more than $52,000. This base rate and accordingly the gross income may be lowered progressively as the sinking-fund is applied in the redemption of the original investment.

In many instances the base rate thus obtained should be corrected by some amount corresponding to the losses of water unavoidably taking place between the source of water-supply and the faucet of the consumer, particularly where the use of pumps is necessary in the distribution of the water service. These losses between the pumps and the faucet of a consumer often amount to fully 25 per cent of the water registered as delivered by the pumps. In other words, for every 1000 gallons of water, pump delivery, there is 750 gallons, faucet delivery. Accordingly a base rate per million gallons, estimated upon the basis of pump delivery, should often be increased by about one-third in order to get a proper value for the water service per million gallons at the faucet.

Where the law provides that the property shall be taxed for the interest and sinking-fund of the bonds issued in construction or purchase of water-works, the gross income should exceed the cost of production by an amount equivalent to from $1\frac{1}{2}$ to 2 per cent of the investment, in order to provide the surplus usually needed to a greater or less extent in meeting contingent expenses of operation and in the repairs, reinforcement, and maintenance of the water-works property.

The foregoing illustrations are further intended to caution those who may have occasion to use the table relating to the cost of producing the water service to do so guardedly, in view of the fact that they represent general averages and that the tabulated computations must receive proper correction in order to render them applicable to a specific case. In this manner they may prove of assistance to the judgment, when definite data of a similar character is not available.

In summing up what has been said about water rates, it may be said that the base rate should embrace the following essential items, cost of production, interest on investment, repairs, sinking-fund, unavoidable losses of water, and a surplus of an amount or percentage depending upon the particular character of the system of water-works under consideration.

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